MODULUS OF ELASTICITY AS A FACTOR IN THE DESIGN OF BITUMINOUS PAVEMENT MIXTURES

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MODULUS OF ELASTICITY AS A FACTOR IN THE DESIGN

OF BITUMINOUS PAVEMENT MIXTURES

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SUMMARY

The purpose of this research is to determine the effect of modulus of elasticity on the quality of bituminous pavement mixtures. The main intent is to introduce flexibility as a factor in the design of bituminous mixtures along with stability and durability.

Flexibility can be measured in several ways. Common methods are plate load tests, repeated loading of laboratory constructed slabs and beams, and measurement of deflections under wheel loads on pavements in test pits or test roads. The modulus of elasticity is also an inverse measure of the flexibility of a material. This property of material, which can be obtained from a stress-strain curve, was used as the flexibility parameter in this research.

A discussion of the theoretical effect of modulus of elasticity on bituminous pavements is presented. This involves a review of the Boussinesq and Burmister theories of stress distribution along with findings of some recent research at Georgia Tech by G. F. Sowers and A. B. Vesic. It was found that stresses in a number of flexible pavement materials followed the Boussinesq theory rather than the layered system theory. The Boussinesq and Burmister equations for deflection are also presented.

The experimental work involved testing prototype specimens of four bituminous pavements which were built in Georgia between 1947 and 1954. Two of the pavements were built with crushed limestone, and two were built with crushed granite; one pavement of each type above was built with local sand. The pavements were originally designed by the Georgia Highway

Department using the Hubbard Field Method. Identical aggregates and grades of asphalt as were used in the original design were used in making the laboratory specimens, and all specimens were compacted using a kneading compactor to densities which were approximately the same as the pavement had initially. These specimens were made in three groups:

1. Relative stability specimens tested in Hveem stabilometer.
2. Relative stability immersion-compression specimens tested in Hveem stabilometer.
3. Triaxial shear specimens.

A comparison was made between the values of stability, angle of internal friction, apparent cohesion, and modulus of elasticity and performance of the four pavements. This comparison revealed that relative stability did not agree well with performance, and that the angle of internal friction and apparent cohesion seemed to give no indication of performance. The modulus of elasticity values showed a good correlation with performance. Higher values of modulus of elasticity were associated with poor pavement performance and lower values were associated with good performance.

CHAPTER I

INTRODUCTION

Current practice in bituminous pavement mix design varies widely. Generally, however, emphasis is placed on two factors: stability and durability. These factors are certainly important in the design of bituminous mixtures. However, there are other factors which should be considered (1, 13). It is difficult to determine which factors are most important but a number of investigators (4, 5, 9) have suggested that flexibility should be considered as one of the more important factors.

Bituminous pavements have long been referred to as "flexible" pavements. This implies that bituminous pavements are capable of undergoing relatively large deformations without distress. If, however, a bituminous pavement should have a very large value of modulus of elasticity (stiffness), it would tend to act like a beam instead of exhibiting flexible characteristics. On the other hand, if the modulus of elasticity should be very low, the deflections of the pavement system may be intolerable. Therefore, it seems that an acceptable range of values of modulus of elasticity should be determined for bituminous pavements. As far as this writer has been able to determine, no such design criteria have been suggested.

An approach to the determination of the influence of modulus of elasticity on bituminous pavements was obtained from this study. The approach is based on a comparison of laboratory test results with observed pavement performance. A theoretical effect is also discussed.

Four bituminous pavements which were built in Georgia between 1947 and 1954 were selected for study. These pavements are listed in Table 1 and additional data are presented in Appendix A. The mixtures were originally designed by the Georgia Highway Department using the Hubbard-Field method.

Table 1. Location and Description of Pavements

Pavement	Location	Year Built	Type Asphaltic Concrete Used	General Performance
I	US 41 from Marietta Georgia to Bartow County Line.	1948-1949	E Surface B Binder	Overall performance is good. Small amount rutting.
II	US 41 from Bartow County Line to Emerson, Ga.	1954-1955	E Surface B Binder	Poor. Shoving badly especially on grades.
III	US 41 one mile north of where 4-lane pavement ends to Gordon County Line.	1947	E Surface B Binder	Poor. Lack of bond between pavement and base. Pot holes.
IV	US 247 from Macon, Georgia to Warner Robins, Georgia	1954	E Surface B Binder	Excellent.

CHAPTER II

THEORETICAL EFFECT OF MODULUS OF ELASTICITY

General.--The modulus of elasticity is a basic property of material used in structural design and analysis. The influence of modulus of elasticity is felt in the calculation of deformation and load distribution in all structural systems, including soil and pavement systems.

Figure 1 shows a typical flexible pavement in cross section. The pavement is constructed in layers, each of which is relatively uniform throughout. Most bituminous pavements exhibit modulus of elasticity values decreasing from layer to layer down through the pavement. This is due to the fact that the top layer is usually very dense and successive lower layers are more open graded, thus having lower values of modulus of elasticity.

Stress Distribution.--The theory of elasticity as applied to a semi-infinite, homogeneous isotropic soil mass by Boussinesq offers a basis for deriving stress distributions under a load in pavement systems. Boussinesq assumed an elastic material with constant values of modulus of elasticity (E) and Poisson's ratio (ν) throughout the mass. The Boussinesq formula for vertical stress at a point below a concentrated load applied at the surface of the soil mass is as follows:

$$\sigma_z = \frac{Q}{z^2} \frac{3}{2} \left[\frac{1}{1 + (r/z)^2}\right]^{5/2} \qquad (1)$$

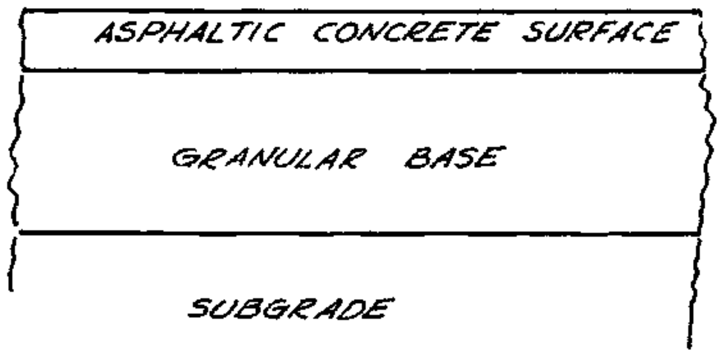


Figure 1. Typical Flexible Pavement.

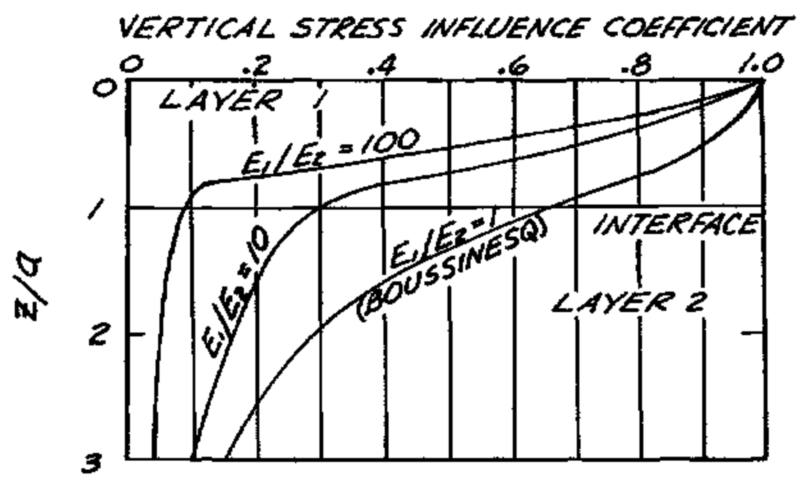


Figure 2. Influence Curves for Stresses in Two Layer System.

where σ_z = vertical stress

Q = concentrated load

z = depth

r = radial distance from load.

It should be noted that equation (1) is dependent only on the total load, the depth, and the radial distance from the load. Modulus of elasticity does not affect the stress distribution.

Equation (1) has been integrated over a circular area with a uniform pressure and the following expressions derived (12):

$$\sigma_z = q\left[1 - \frac{z^3}{(a^2+z^2)^{\frac{3}{2}}}\right] \tag{2}$$

$$\sigma_r = \frac{q}{2}\left[1 + 2\nu - \frac{2(1+\nu)z}{(a^2+z^2)^{\frac{3}{2}}} + \frac{z^3}{(a^2+z^2)^{\frac{3}{2}}}\right] \tag{3}$$

where σ_z = vertical stress at points beneath the center of the loaded area

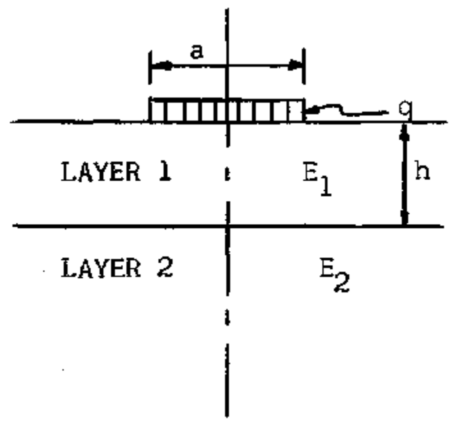
σ_r = radial stress at points beneath the center of the loaded area

z = depth

a = radius of the loaded area

ν = Poisson's ratio.

The Boussinesq theory was later extended by Burmister (8) to accommodate layered systems. Burmister reasoned that if layers in a soil mass had different values of modulus of elasticity the stress distribution would be different from that given by Boussinesq (equations (2) and (3)). If the upper layer is more rigid than the lower layer (i.e. higher value of modulus of elasticity) then the stresses would be reduced at a greater rate than that given by equation (2). This would be due to beam action in the upper layer. The mathematical representation of Burmister's theory is very complicated and is usually represented graphically as shown in Figure 2. It can be seen from Figure 2 that stress at any depth decreases with an increase in the ratio E_1/E_2.



The primary assumptions of the Burmister theory are as follows:

1. Each layer is homogeneous and isotropic
2. Each layer is infinite in horizontal direction
3. Interfaces are perfectly rough

Burmister's theory of the layered system stress distribution has found application in flexible pavement system analysis. If the layers of a flexible pavement system could be so designed as to obtain values of modulus of elasticity decreasing with depth, then it might be reasoned stresses would be reduced much faster than if all layers had the same value of modulus of elasticity. This procedure could possible result in designing the expensive base and surface to a smaller thickness than if the Boussinesq theory is assumed to apply. However, according to research conducted at Georgia Tech by G. F. Sowers and A. B. Vesic (7), this theory was discredited for a number of bases and surface materials. It was found that in pavements with sand asphalt, granular, or topsoil bases and asphaltic concrete surfaces, the stress distribution in the base and subgrade more nearly followed the Boussinesq theory than the two layer theory. This apparent contradiction to theory is explained by the fact that since all of the above materials exhibit very little cohesion, the relatively stiff upper layer cannot transfer the high tensile stresses developed at the interface. This condition of course places the problem outside of the scope of Burmister's theory which assumes a material equally strong in tension and compression.

Deflections.--Expressions for elastic deflections for both the Boussinesq and Burmister theories have been presented (8, 12). The Boussinesq is derived as follows:

$$\varepsilon_z = \frac{\partial \delta_z}{\partial z} = \frac{1}{E}\left(\sigma_z - 2\nu\sigma_r\right) \qquad (4)$$

where ε_z = vertical strain

z = depth

δ_z = vertical deflection

σ_z = vertical stress from equation (2)

σ_r = radial stress from equation (3)

E = modulus of elasticity

ν = Poisson's ratio.

Substituting equations (2) and (3) into equation (4) and integrating between limits z and ∞ gives,

$$\delta_z = \frac{q}{E}\left[\left(2-2v^2\right)\left(r^2+z^2\right)^{1/2} - \frac{(1+v)z^2}{\left(r^2+z^2\right)^{\frac{1}{2}}} + \left(v+2v^2-1\right)z\right] \tag{5}$$

where symbols have the same definitions as above.

The Burmister expressions for deflections are as follows:

$$\delta_z = \frac{1.5qr}{E_2} F_2 \text{ (rigid plate)} \tag{6a}$$

$$\delta_z = \frac{1.18qr}{E_2} F_2 \text{ (flexible plate)} \tag{6b}$$

where δ_z = vertical deflection

q = uniform pressure on circular area

r = radial distance from center of loaded area

E_2 = modulus of elasticity of lower layer

F_2 = dimensionless factor depending on $\frac{h}{r}$ and $\frac{E_1}{E_2}$ (see Figure 3).

It can be seen from the foregoing discussion that the modulus of elasticity plays a definite role in deflection calculations using both the Boussinesq and Burmister methods. Deflections vary inversely as the modulus of elasticity.

In order to reduce deflections, a high value of modulus of elasticity would be desirable. However, an upper limit on the stiffness of the pavement system is suggested (9) to insure adequate flexibility. This upper limit is necessary to afford the pavement satisfactory fatigue characteristics. The visco-elastic properties of asphaltic concrete cause the rebound rate after application of load to be dependent on time. Under repeated application of load, an asphalt pavement will retain small amounts

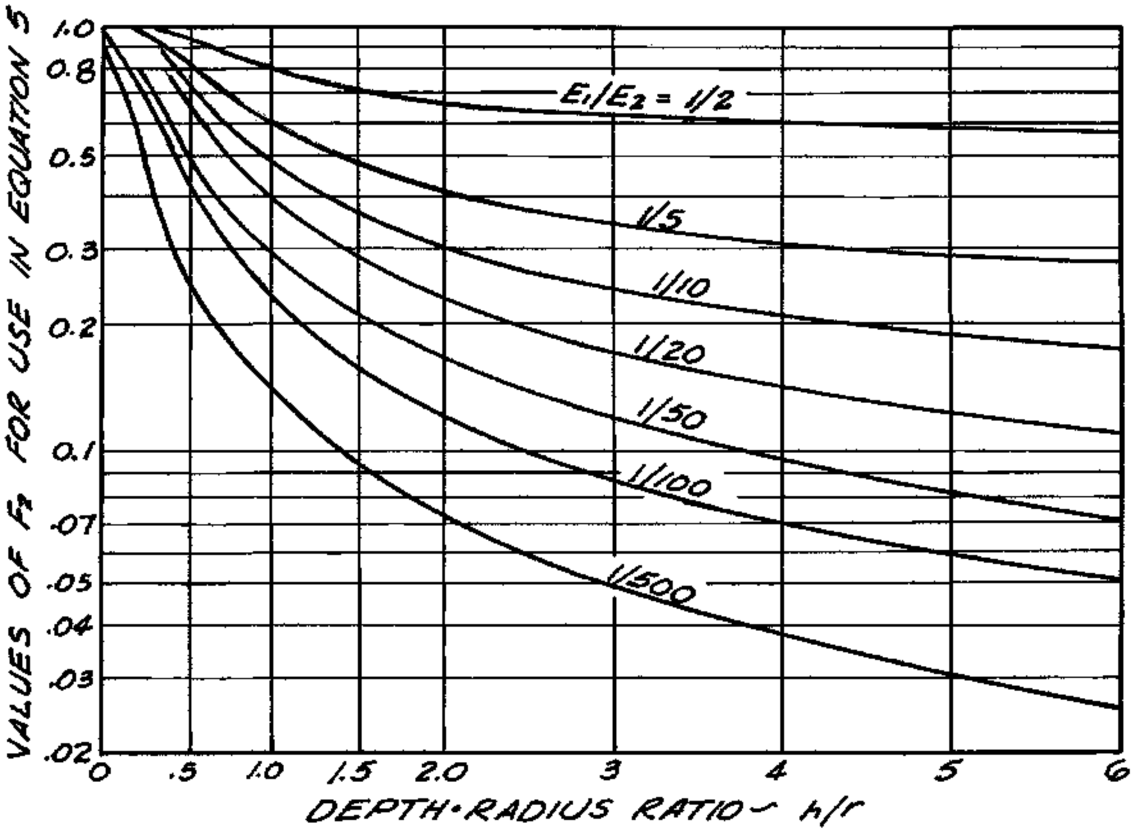


Figure 3. Influence Factors for Two Layer Deflection Calculations.

of set and successive deflections become greater. The result is a fatigue failure and this can occur at very low loads (4).

Summary.--The effect of modulus of elasticity on bituminous pavements is grouped into two catagories: stress distribution and deflections.

(1) Stress distribution - Modulus of elasticity does not appreciably affect stress distribution. Bituminous pavement mixtures tend to distribute stresses similar to an ideal soil mass (Boussinesq).

(2) Deflections - The deflection is inversely proportional to the modulus of elasticity.

(3) Extremely high values of modulus of elasticity could cause the pavement to fail prematurely in fatigue.

CHAPTER III

TEST PROGRAM

General.--Prototype specimens of the four pavements listed in Table 1 were tested to determine the following parameters:

(1) Relative Stability

(2) Shear Strength

(3) Modulus of Elasticity

Relative stability was determined for the pavement mixtures using the Hveem Stabilometer on specimens 2-1/2 inches high and 4 inches in diameter. These specimens were tested in two groups: regular specimens and immersion-compression. The latter group gives an indication of the durability of the mixtures.

Shear strength and modulus of elasticity were determined from triaxial shear test data. Specimens 8-1/2 inches high and 4 inches in diameter were tested.

Material and Equipment.--The aggregates used for this research were the same as those used in the original design and construction of each pavement. These are listed in Appendix A. The crushed stones were obtained from the original quarries and the sands were obtained from the original borrow pits by the Georgia Highway Department. The asphalt cement, AC8, was obtained from American Bitumal Company.

The equipment used was as follows:

1. Oven at 300° F
2. Oven at 230° F

3. Oven at 140° F
4. Constant temperature water bath at 140° F
5. Hobart Models C-100 and N-50 mixers
6. Mixing bowls, trawels, spoons, etc.
7. Soil test CN-425A kneading compactor with heated foot
8. Hveem stabilometer
9. Triaxial cell capable of accommodating 4 inch diameter specimen
10. Constant strain load machine

Compaction Procedures.--All specimens were compacted to a density which represents the pavement as it was in its initial condition. The compaction procedures to give these densities were developed in previous research conducted at Georgia Tech (1).

The relative stability specimens were compacted by the following procedure:

1. After heating the aggregate and asphalt to a temperature of 300° F, the proper amount of asphalt was added to each batch and the mix thoroughly mixed using a Hobart N-50 mechanical mixer.

2. The mix was placed in a feeder trough 18 inches long, 4 inches wide, and 2-1/2 inches deep.

3. The mold was placed in mold holder with 1/4 inch shim under mold.

4. The mix was fed into the mold in three layers each of which was rodded with a 3/8 inch bullet nosed steel rod, 20 times in the center and 20 times around the edge.

5. The mold with mix was then placed on compactor base and 15 tamping blows at 250 psi foot pressure were applied. The dwell period was 0.4 seconds.

6. The 1/4 inch shim was removed and 35 additional tamping blows were applied at 250 psi foot pressure with dwell period of 0.4 seconds.

7. The mold with the compacted specimen was then placed in an oven at 140° F for three hours. A leveling load of 1000 psi was then applied using a double plunger.

8. The specimens were removed from the molds.

Triaxial specimens were compacted according to the following procedure:

1. After heating the aggregate and asphalt to a temperature of 300° F, the proper amount of asphalt was added to each batch and the mix throughly mixed, using a Hobart C-100 mechanical mixer.

2. The mix was placed in eight aluminum trays 2 inches wide, 1-1/2 inches deep and 13 inches long and placed in an oven at 230° F until the mix temperature stabilized at 230° F.

3. With mold and mold holder on compactor base, the mix was fed into the mold from each of the aluminum trays in increments. Each increment was subjected to a number of tamping blows as shown in Table 2. The dwell period was 0.4 seconds.

4. The specimen was removed from the mold and placed in an oven at 140° F for three hours. A leveling load of 6300 pounds (500 psi) was applied using a double plunger.

<u>Relative Stability Testing</u>.--The stabilometer test was performed in the manner prescribed by the California Highway Department (14). Lateral pressures corresponding to vertical pressures are recorded and the displacement of the specimen is measured by means of a displacement pump on the stabilometer. Relative stability was calculated by the expression

Table 2. Compaction Schedule for Triaxial Specimens

Line Air Pressure (PSI)	Foot Pressure (PSI)	No. Blows	Portion of Mix Placed in Mold (Trays)
		10	1
		15	2
		15	3
10.0	81.0	14	4
		14	5
		14	6
		14	7
		14	8
		110	
5	40.5	15	
		125	

$$S = \frac{22.2}{\frac{PhD}{Pv-Ph} + .222}$$

where S = relative stability

Ph = lateral pressure corresponding to Pv = 400 psi

Pv = vertical pressure = 400 psi

D = displacement.

Immersion - Compression Specimens.--In addition to the regular stability specimens, identical specimens were placed in a constant temperature water bath at 140° F for 24 hours prior to testing. After the 24 hour immersion period, the specimens were tested for stability in the same manner as regular specimens. The difference in stability between regular and immersion-compression specimens serves as an indication of the durability of the mix.

Triaxial Shear Testing.--All triaxial specimens were tested at room temperature (75° ± 2° F). In order to reduce the effect of viscosity of asphalt the tests were run at a rate of deformation of 0.01 inch per minute. Confining pressures of zero, 20 psi, and 40 psi were used. After applying the proper confining pressure, loads corresponding to deformations taken at 0.025 inch interval are recorded. From the data thus obtained, a stress-strain curve was plotted from which the modulus of elasticity was taken (see Appendix B). From the values of maximum stress at each confining pressure (average of two specimens), a Mohr envelope was constructed for each mixture and the angle of internal friction and apparent cohesion determined (see Appendix B).

CHAPTER IV

TEST RESULTS

Relative Stability.--Results of the relative stability tests are reported in Table 3 and presented graphically in Figures C1 through C4 in Appendix C. Figures C1 through C4 show the variation in relative stability with asphalt content for both regular and immersion-compression specimens.

Relative stability is sensitive to asphalt content as seen from the figures in Appendix C. All mixtures except the E surface for Pavement III exhibit the same general shape stability curve, concave downward and decreasing with increased asphalt content. This is true of regular and immersion-compression specimens. The immersion-compression curves tend to run parallel and below those for regular specimens. There does not seem to be any strong indication that loss in stability due to immersion is curtailed by the addition of asphalt as one might expect.

Triaxial Shear Tests.--Results of the triaxial shear tests are reported in Table 4 and presented graphically in Figures D1 through D8 in Appendix D. The values of angle of internal friction, apparent cohesion and modulus of elasticity are reported. Included in Table 4 are values of percent strain at failure. Figures D1 through D8 show the variation of these parameters with asphalt content.

The angle of internal friction and apparent cohesion vary erratically for the mixtures studied. For example Pavement I, E surface, had an angle of internal friction which was nearly constant and a cohesion increasing with asphalt content. Pavement II, E surface, had an angle of internal

Table 3. Hveem Stability Test Results

Pavement	Type Mix	Percent Asphalt	Relative Stability: Regular Specimens	Relative Stability: Immersion-Compression Specimens
I	E Surface	6	34.1	28.5
		7	32.5	28.3
		8	23.3	14.1
	B Binder	5	36.2	27.2
		5 3/4	30.6	21.8
		7	8.1	3.1
II	E Surface	6	38.0	26.7
		7 1/4	29.8	23.8
		8	24.7	22.3
	B Binder	4	42.4	32.6
		4 3/4	36.8	28.9
		6	30.8	19.7
III	E Surface	6	21.5	24.6
		7	11.6	5.9
		8	0	0
	B Binder	4	31.6	22.3
		4 3/4	32.6	25.6
		6	25.0	16.7
IV	E Surface	5.5	28.8	27.1
		6.5	22.9	23.8
		7.5	10.6	7.8
	B Binder	4	27.2	28.3
		5	24.2	28.4
		6	24.4	18.2

Table 4. Triaxial Test Results

Pavement	Type Mix	Percent Asphalt	Internal Friction Angle (DEG)	Unit Cohesion (PSI)	Modulus of Elasticity* (KSI)		
					$\sigma_3 = 0$	$\sigma_3 = 20$	$\sigma_3 = 40$
	E	6	42.6	12.0	5.40 (1.4)	11.50 (2.8)	15.55 (3.5)
I	Surface	7	39.8	18.0	7.00 (1.5)	12.35 (2.8)	13.30 (3.8)
		8	40.7	20.0	6.13 (1.9)	9.25 (3.4)	12.70 (4.5)
	B	5	39.3	16	8.10 (1.1)	13.40 (2.9)	14.40 (6.4)
	Binder	5 3/4	40.4	17	7.40 (1.5)	12.70 (4.3)	14.20 (4.9)
		7	33.4	21	4.90 (2.5)	7.85 (6.1)	9.80 (6.2)
	E	6	39.8	25.0	10.40 (1.3)	15.10 (2.4)	18.55 (3.6)
II	Surface	7 1/4	39.5	27.0	10.45 (1.7)	14.25 (2.5)	16.30 (3.2)
		8	39.0	22	10.20 (1.6)	12.50 (3.6)	16.35 (4.1)
	B	4	43.5	11	7.40 (1.3)	11.70 (2.8)	16.00 (5.4)
	Binder	4 3/4	42.9	11	7.65 (.9)	14.35 (2.6)	16.70 (4.7)
		6	43.8	11	6.75 (1.5)	14.90 (3.4)	23.80 (5.3)
	E	6	36.1	16.5	10.35 (1.2)	15.95 (3.6)	21.10 (4.8)
III	Surface	7	39.4	16.0	5.85 (2.1)	14.50 (5.1)	16.90 (6.1)
		8	39.0	15.0	4.65 (3.3)	9.40 (6.7)	6.30 (10.1)
	B	4	39.3	11	4.50 (1.6)	10.40 (3.1)	17.60 (2.9)
	Binder	4 3/4	37.2	14	7.60 (1.4)	13.90 (3.5)	23.00 (3.1)
		6	25	32	3.00 (1.4)	9.00 (3.6)	15.60 (2.8)
	E	5.5	37.9	17	10.99 (1.1)	15.34 (1.6)	17.39 (2.3)
IV	Surface	6.5	36.1	20	7.42 (1.4)	11.92 (2.1)	12.85 (3.0)
		7.5	37.9	13	1.70 (3.8)	7.67 (4.0)	5.58 (4.6)
	B	4	42.3	11	6.50 (1.0)	16.30 (1.6)	24.20 (2.4)
	Binder	5	41.0	13	7.45 (1.1)	12.50 (1.9)	17.45 (3.1)
		6	37.6	15	3.60 (2.5)	6.20 (3.4)	6.40 (7.4)

*Numbers in parentheses are values of unit strain at failure in percent.

friction which was constant but cohesion decreased with increased asphalt. Pavement III, B binder, had an angle of internal friction decreasing with increased asphalt and cohesion increasing with asphalt. Generally, however, it can be concluded that the angle of internal friction remains relatively constant throughout the range of asphalt contents.

The modulus of elasticity has a very wide range of values. Generally Figures D1 through D8 show that the modulus of elasticity increases with confining pressure and decreases with increased asphalt content.

Evaluation of Test Data.--Table 5 is a tabulation of weighted average values of angle of internal friction, apparent cohesion, relative stability and modulus of elasticity for the pavements at the design asphalt content of each. Included in Table 5 are weighted average values of percent strain at failure. These average values were calculated on the basis of thickness of the respective layers.

A study of Table 5 reveals that the angle of internal friction and apparent cohesion do not appear to be different for the four pavements. There was, however, a significant difference between stability values obtained for Pavements I and II and values obtained for Pavements III and IV. Stability values for Pavements I and II were 31.4 and 33.8 respectively and Pavements III and IV had stability values of 23.6 and 23.7 respectively. This indicates that Pavements I and II should perform well when compared to Pavements III and IV. This, however, was not the case (Table 1). Pavements I and IV had good performance and Pavements II and III had poor performance. This information indicates that for the four pavements studied in this research, relative stability is not a valid criterion for predicting pavement performance.

Table 5. Weighted Average Values of Strength Parameters for Each Pavement at Design Asphalt Content

Pavement	Relative Stability S	Internal Friction Angle Ø	Apparent Cohesion C	Modulus of Elasticity* (KSI) $\sigma_3 = 0$	$\sigma_3 = 20$	$\sigma_3 = 40$
I	31.4	40.2	17.0	7.25 (1.5)	12.57 (3.8)	13.86 (4.5)
II	33.8	41.5	17.9	8.85 (1.2)	14.31 (2.6)	16.53 (4.1)
III	23.6	38.1	14.9	6.85 (1.7)	14.16 (4.2)	20.39 (4.4)
IV	23.7	38.9	16.0	7.44 (1.2)	12.25 (2.0)	15.48 (3.1)

*Numbers in parentheses are values of unit strain at failure in percent.

The modulus of elasticity values of the four pavements do seem to agree with pavement performance to some extent. From Table 5 it can be seen that at confining pressures of 20 psi and 40 psi the values of modulus of elasticity are higher for pavements II and III (which had poor performance), than for pavements I and IV (which had good performance). This information is shown graphically in Figure 4. The values of modulus of elasticity seem to agree with the theoretical idea that generally good performance from a fatigue failure viewpoint should be associated with relatively low moduli of elasticity. This of course does not preclude the fact that very low values of modulus of elasticity could cause deflection failures.

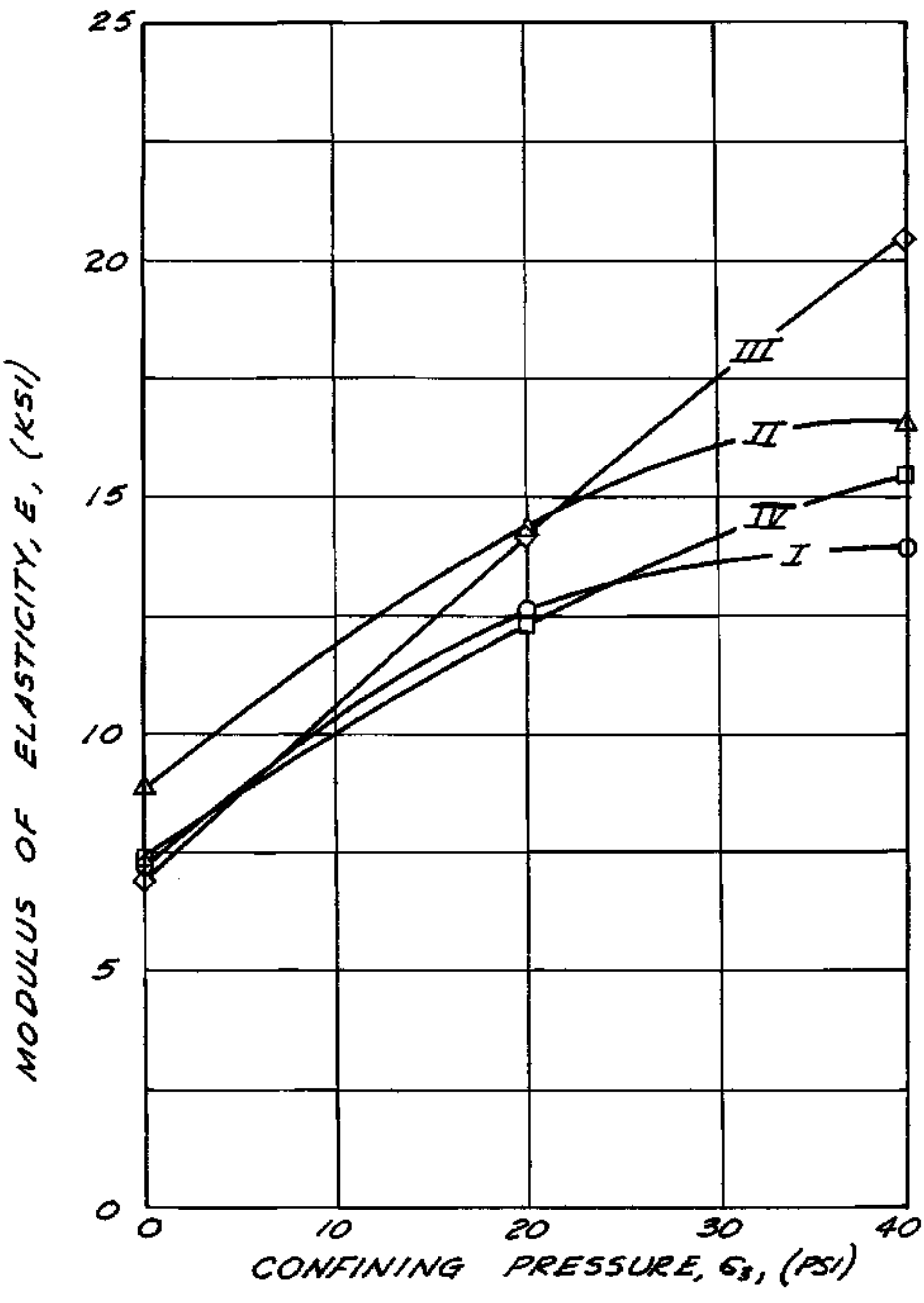


Figure 4. Variation of Weighted Average Values of Modulus of Elasticity with Confining Pressure.

CHAPTER V

SUMMARY AND CONCLUSIONS

Summary.--A summary of test results is as follows:

1. The modulus of elasticity of bituminous paving mixtures increases with confining pressure and decreases with increased asphalt content.

2. From the data obtained for the four pavements studied, a good correlation between relative stability and pavement performance was not possible.

3. The angle of internal friction and apparent cohesion did not change significantly through the range of asphalt contents studied.

Conclusions.--From the test results and the discussion in the previous chapters, the following conclusions are drawn:

1. High values of modulus of elasticity seem to be associated with poor pavement performance and low values with good pavement performance.

2. The angle of internal friction and apparent cohesion do not seem to give an indication of pavement performance.

3. Better correlation between stability and modulus of elasticity should be sought.

4. An acceptable range of values of modulus of elasticity for bituminous pavements should be determined.

APPENDIX A

PAVEMENT DATA

Table A1. Pavement Data

Pavement	Surface Mix	Thickness (IN)	Design Asphalt Content (%)	Grade Asphalt	Type Base	Aggregates
I	E Surface	1 1/2	7	AC 8	Soil Bound Macadam	Crushed stone and screening from Stockbridge Stone Co. Kennesaw, Georgia
	B Binder	2 1/2	5 3/4			
II	E Surface	1 1/2	7	AC 8	Soil Bound Macadam	Limestone and limestone screening from Stockbridge Stone Co. White, Georgia and Etowah River sand
	B Binder	2	4 3/4			
III	E Surface	1 1/2	7	AC 8	Topsoil	Limestone and limestone screening from local quarry, Adairsville, Georgia
	B Binder	2	4 3/4			
IV	E Surface	1 1/2	6.5	AC 8	Topsoil	Crushed granite stone from Weston and Brooker Co. Columbia, S. C., and Granite Hill, Georgia and local sand
	B Binder	2	5			

Table A2. Aggregate Gradation

U. S. Standard Sieve Size	Percent Passing							
	Pavement I		Pavement II		Pavement III		Pavement IV	
	E Surface	B Binder	E Surface	B Binder	E Surface	B Binder	E Surface	B Binder
1		100		100		100		100
3/4		95		80		91	100	97
1/2	100	71		57	100	68	98	81
3/8	98	53	100	42	98	59	85	58
No. 4	76	40	75	37	74	38	63	40
No. 8	59	26	50	30	50	25	50	35
No. 16	45	21	31	16	36	19	47	28
No. 50	25	12	11	10	16	11	21	13
No. 100	15	7	10	6	11	7	10	7
No. 200	8	3	6	4	7	3	5	3

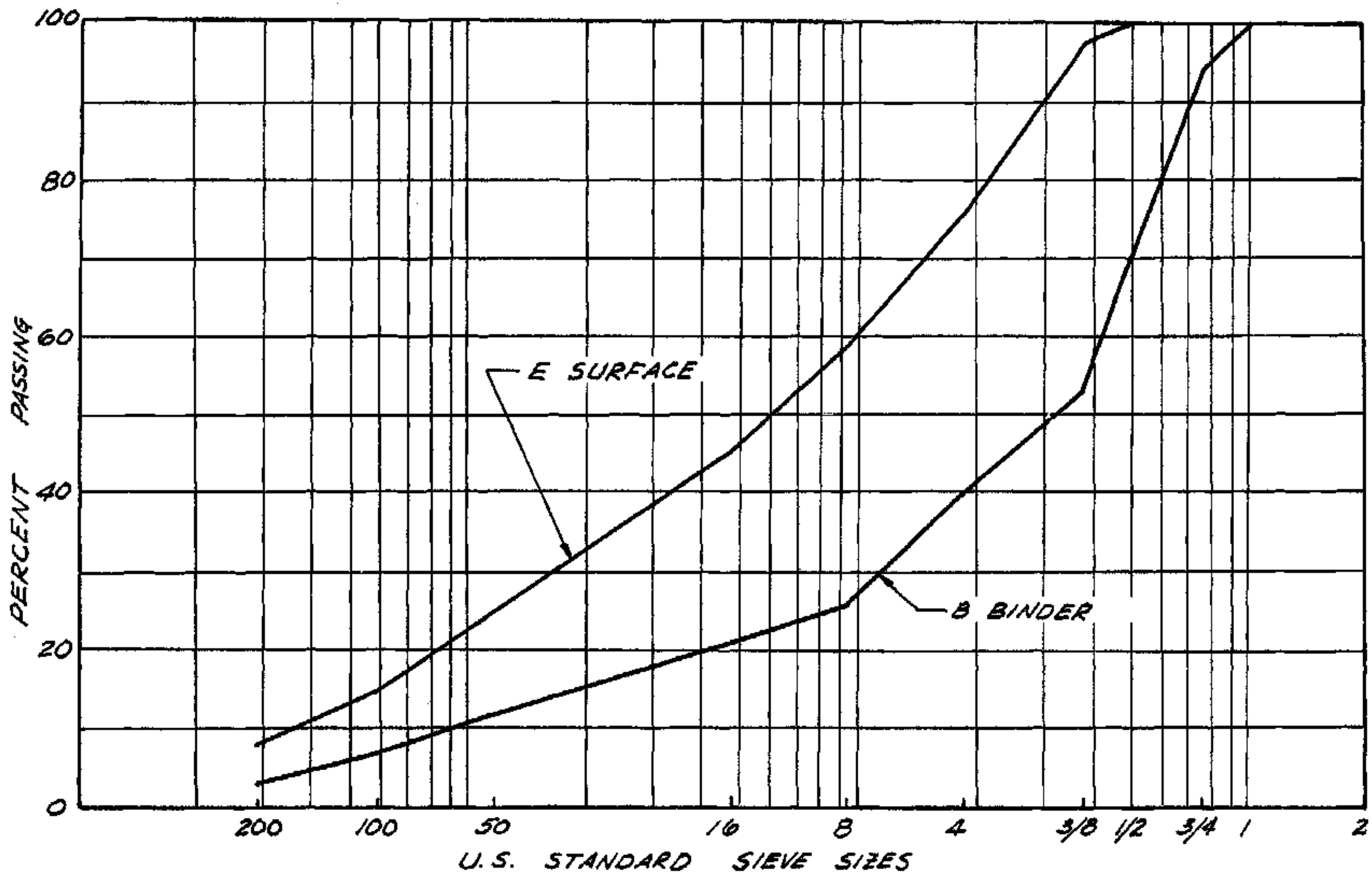


Figure A1. Aggregate Gradation Curves for Pavement I.

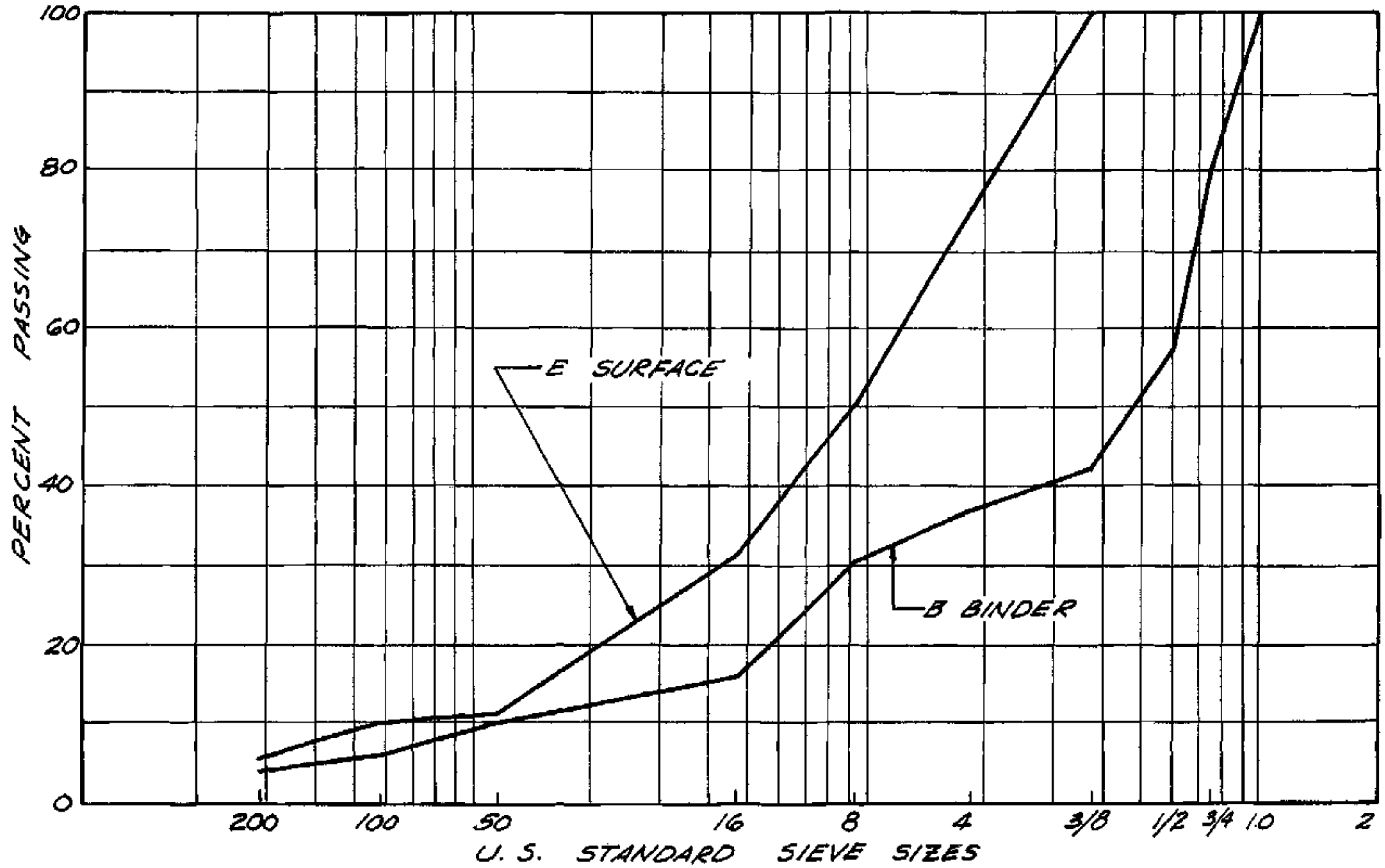


Figure A2. Aggregate Gradation Curves for Pavement II.

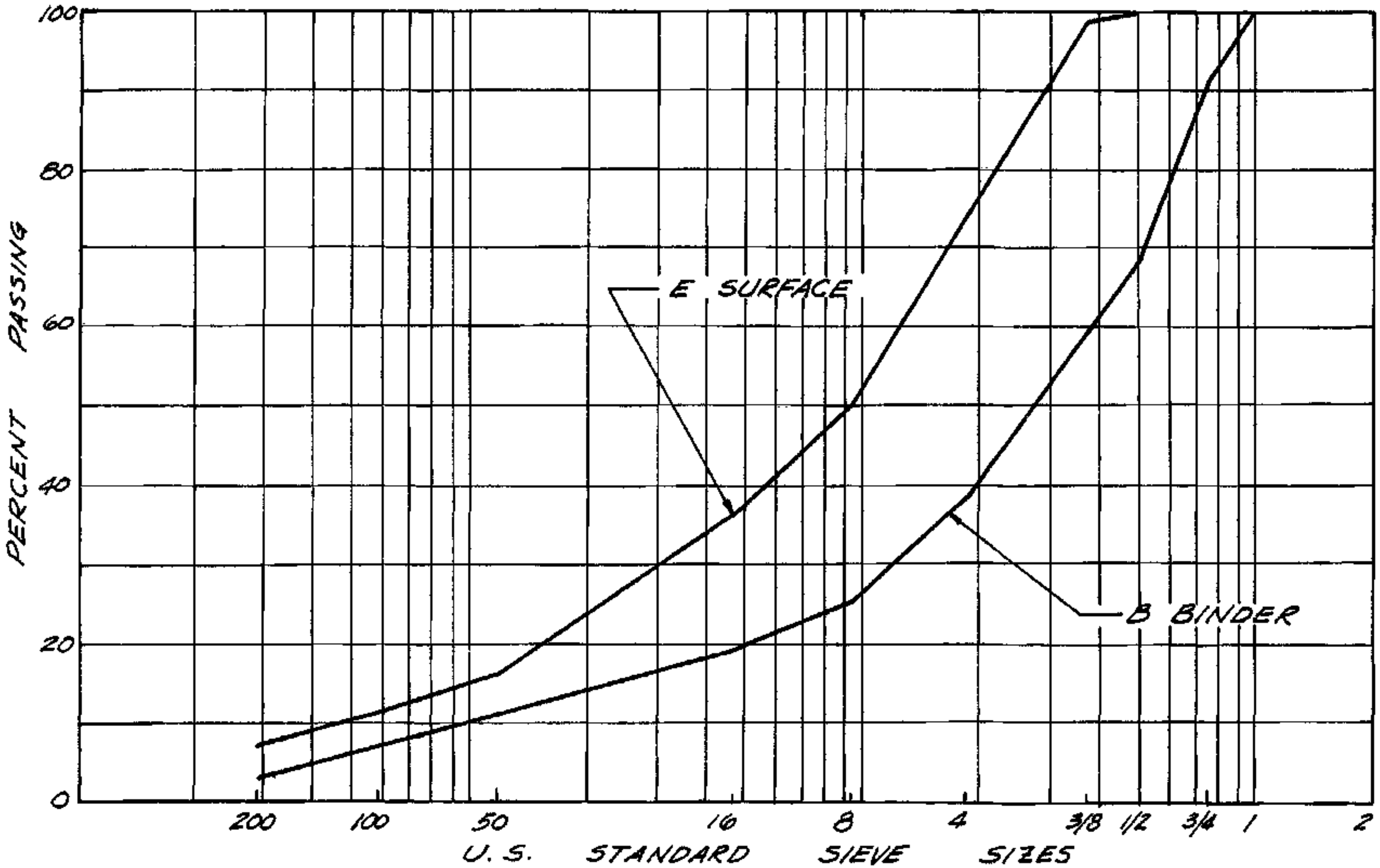


Figure A3. Aggregate Gradation Curves for Pavement III.

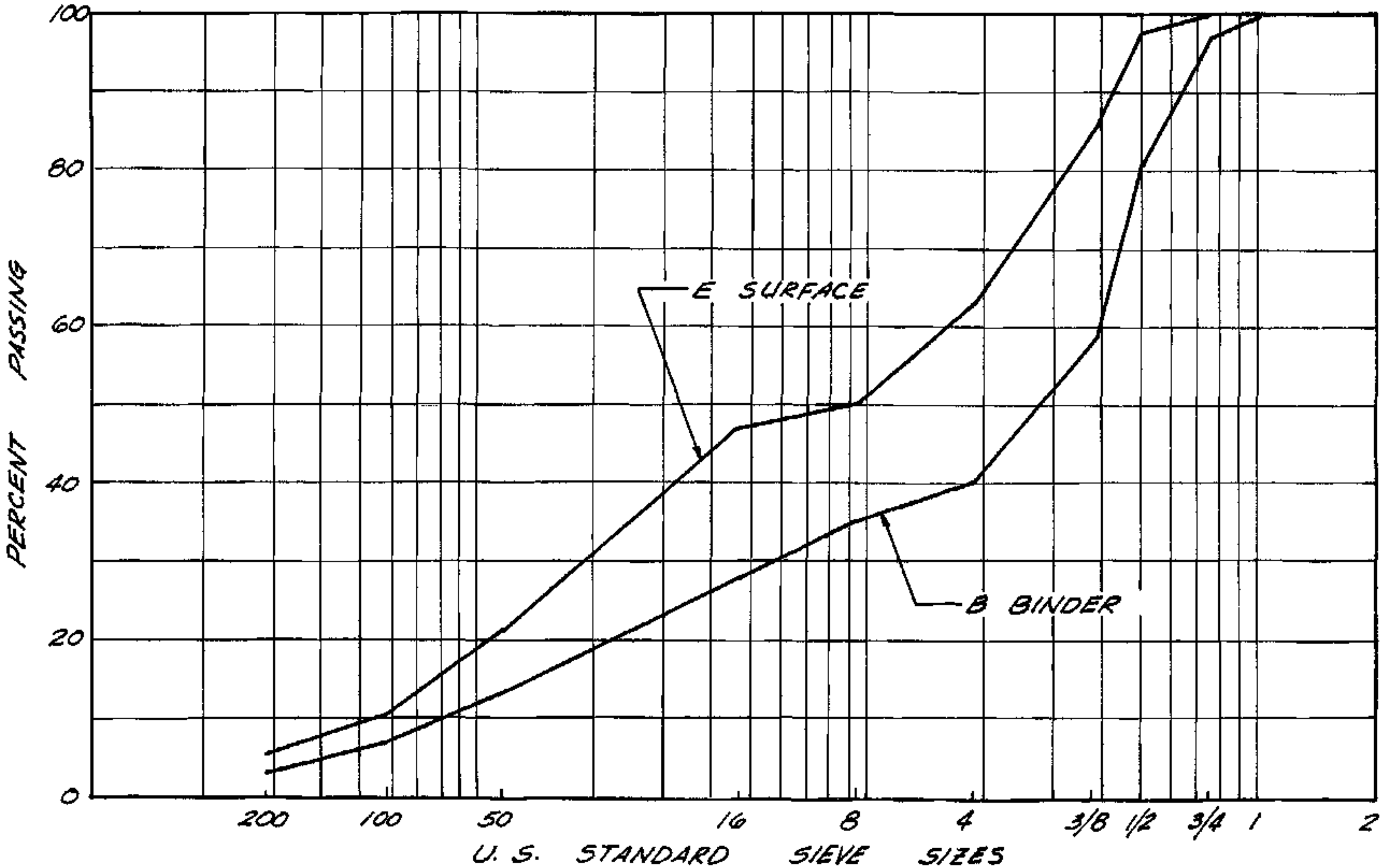


Figure A4. Aggregate Gradation Curves for Pavement IV.

APPENDIX B

COMPUTATIONS FOR TRIAXIAL TEST DATA

COMPUTATIONS FOR TRIAXIAL TEST DATA

Procedure.--From data obtained in the triaxial shear tests, a curve of load vs. deflection was drawn. Figure B1 is a typical curve of this type for the E surface, Pavement I. For each triaxial shear test, such a load-deflection curve was drawn. The curve was projected back to the point of zero load. In most cases this resulted in a correction to the deflections necessary before unit strains could be calculated. This is shown in Figure B1.

After the correction to the deflection was made, unit strains were calculated as follows:

$$\varepsilon = \frac{\Delta h}{h} \times 100\%$$

where ε = unit strain in percent

h = specimen height

Δh = corrected deflection.

The cross sectional area of the specimens were corrected for bulging by the expression

$$A' = \frac{A}{100 - \varepsilon}$$

where A' = corrected area

A = original area

ε = unit strain in percent.

A curve of deviator stress $(\sigma_1 - \sigma_3)$ vs. unit strain was plotted. Figure B2 is typical of this type curve for E surface, Pavement I.

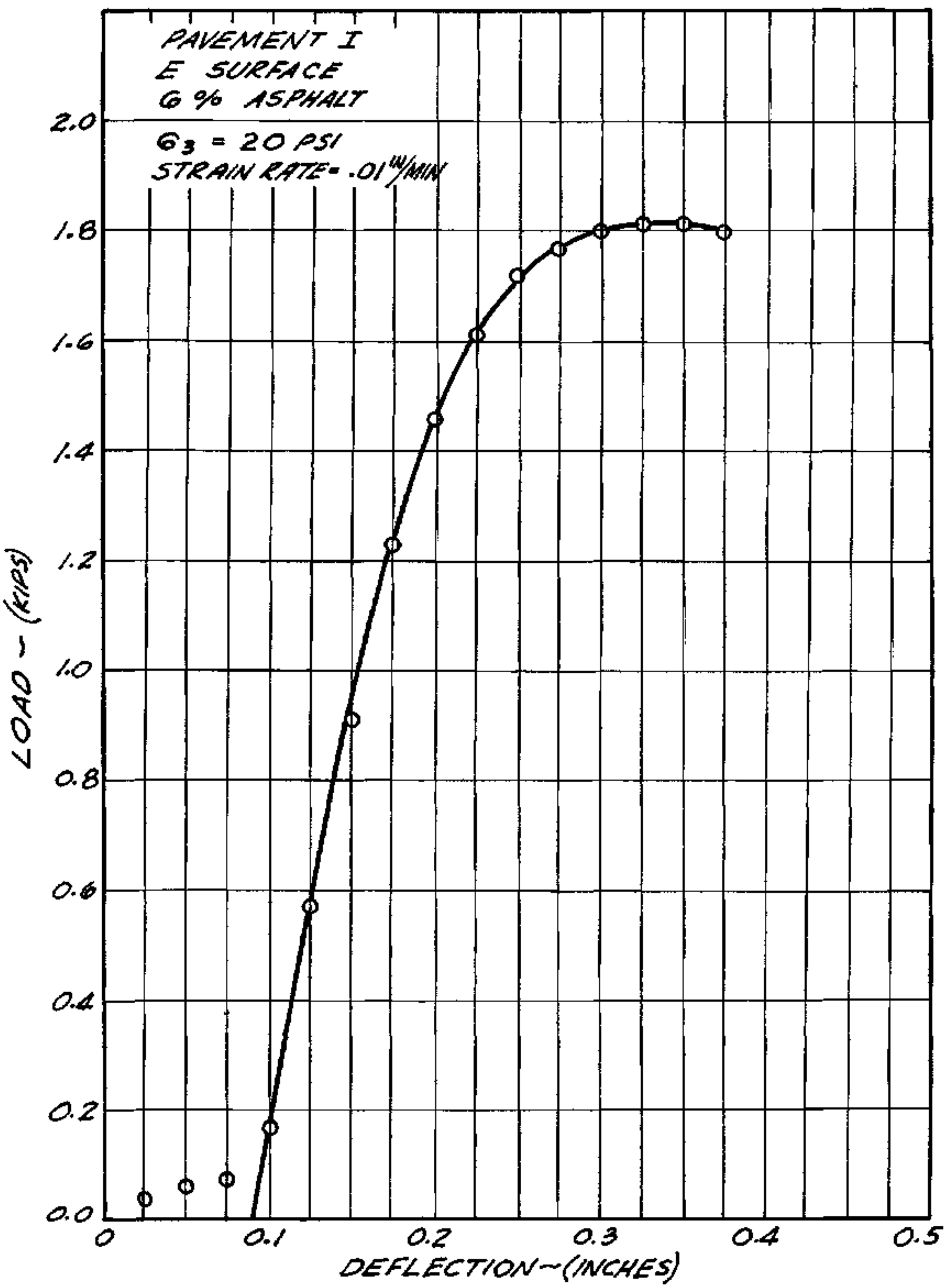


Figure B1. Load-Deformation Curve for E Surface, Pavement I.

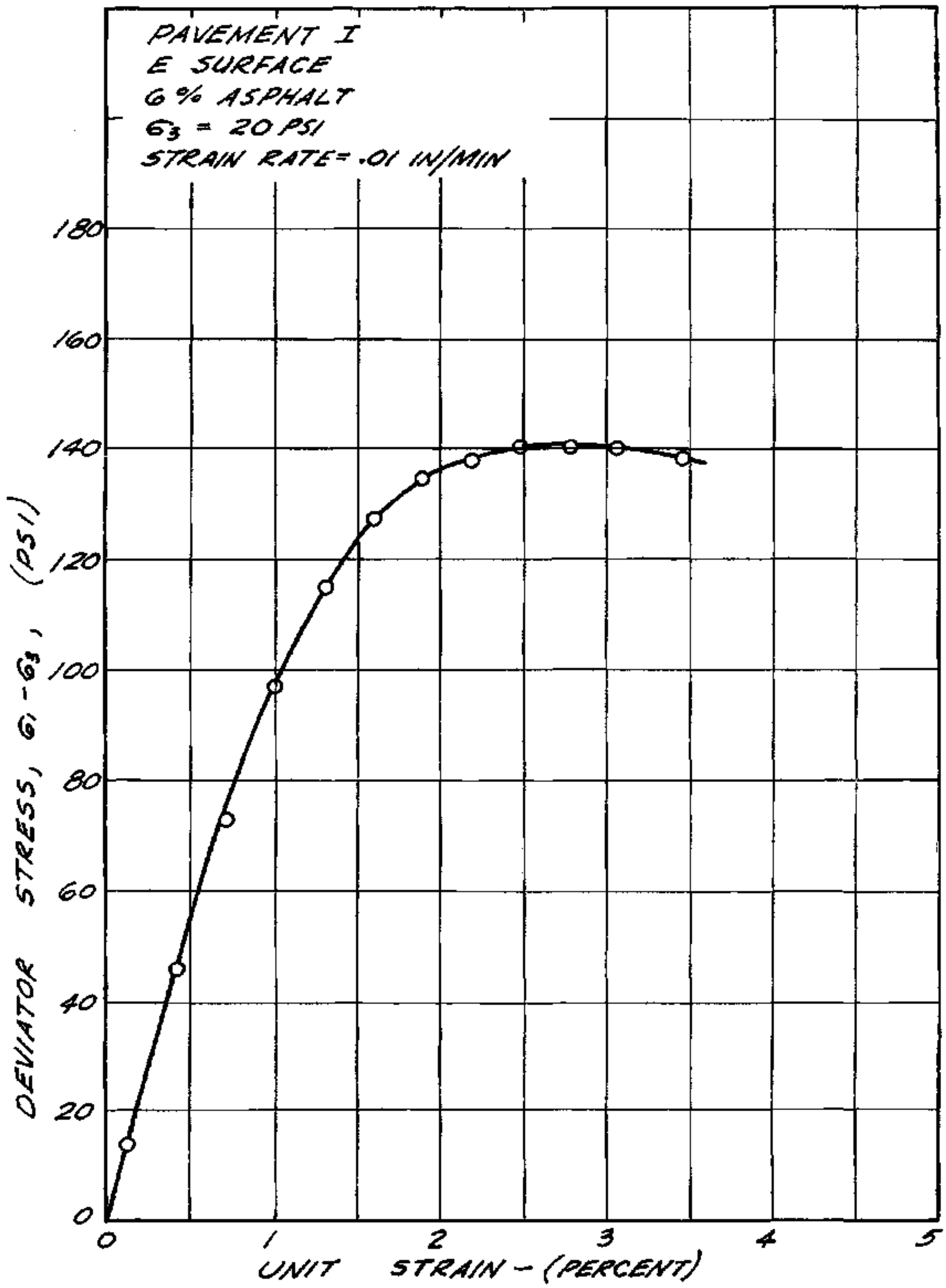


Figure B2. Stress-Strain Curve for E Surface, Pavement I.

The values of deviator stress were calculated as follows:

$$\sigma_1 - \sigma_3 = \frac{P}{A'}$$

where $\sigma_1 - \sigma_3$ = deviator stress

P = load

A' = corrected area.

From the stress-strain curve values of modulus of elasticity were obtained as the tangent to the curve drawn through the point of zero strain.

Mohr Envelope.--With data obtained from the triaxial shear tests, the average value of $(\sigma_1 - \sigma_3)$ was determined for each value of confining pressure (σ_3) and three Mohr Circles were plotted. A line of approximate best fit was then drawn tangent to the circles and established as the Mohr Envelope. An example of such a curve is shown in Figure B3 which is constructed from data taken from triaxial shear tests for the E surface mix, Pavement I, with 6 percent asphalt.

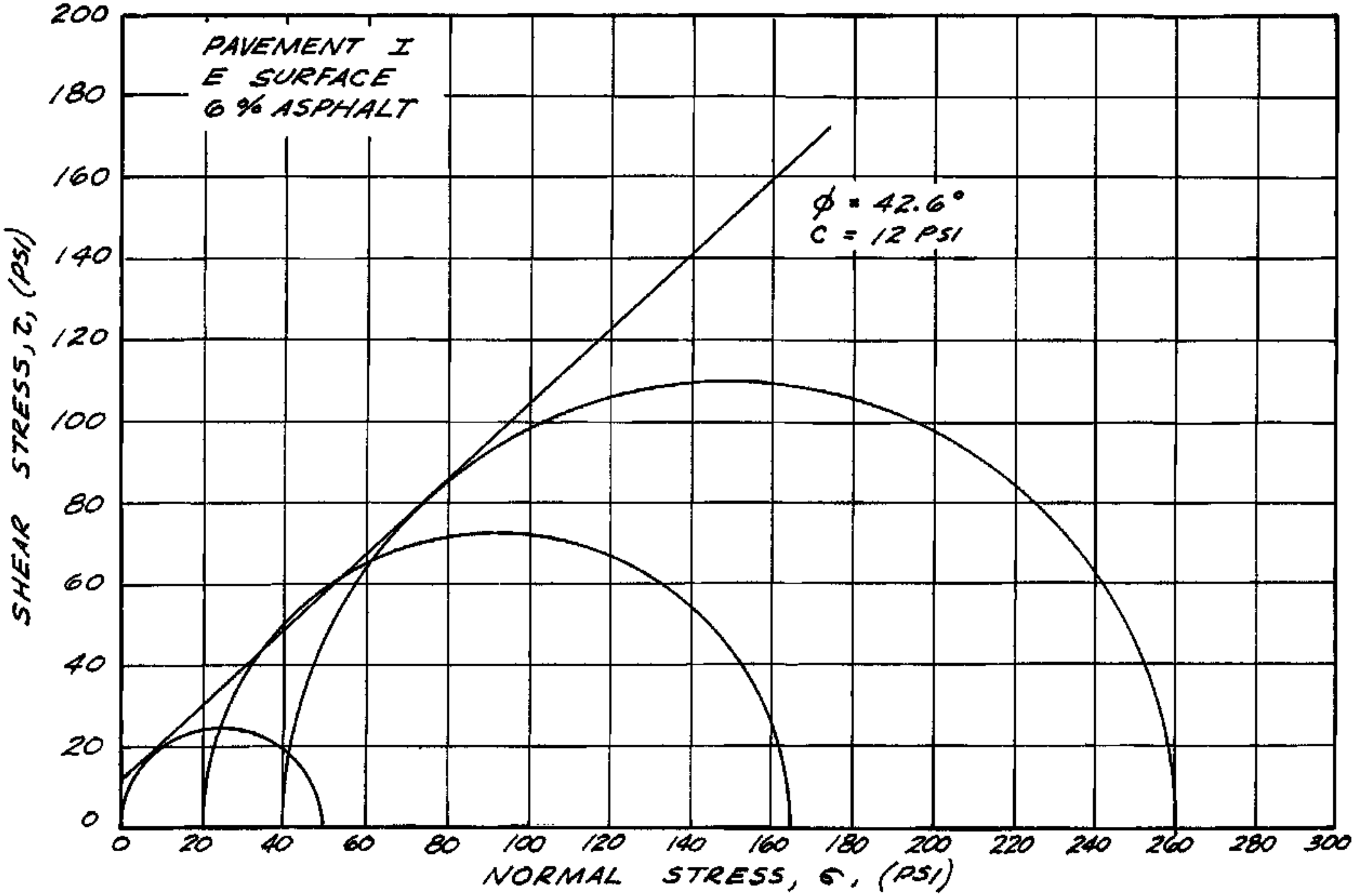


Figure B3. Mohr Envelope for E Surface, Pavement I.

APPENDIX C

RESULTS OF HVEEM STABILITY TESTS

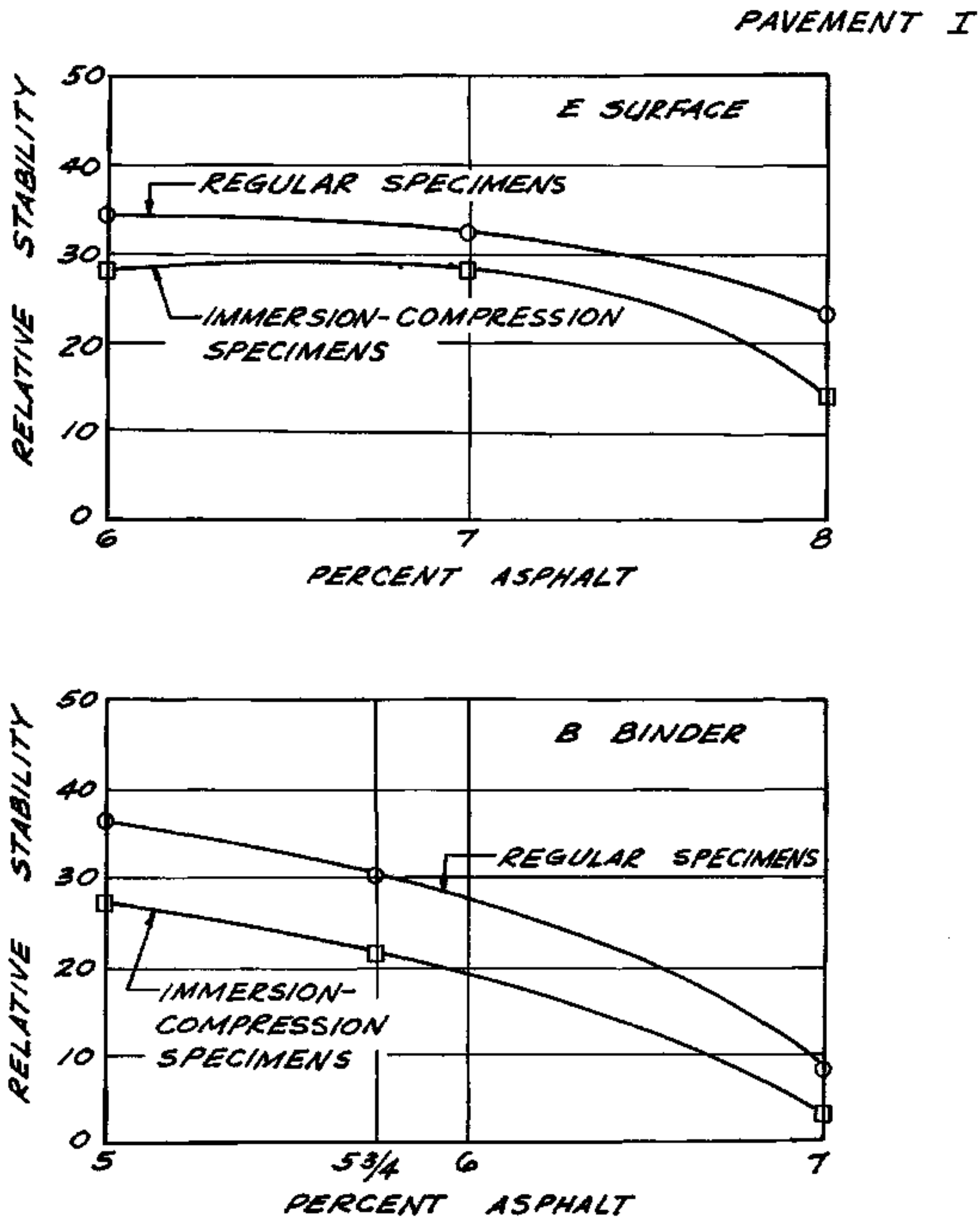


Figure C1. Variation in Relative Stability with Asphalt Content for Pavement I.

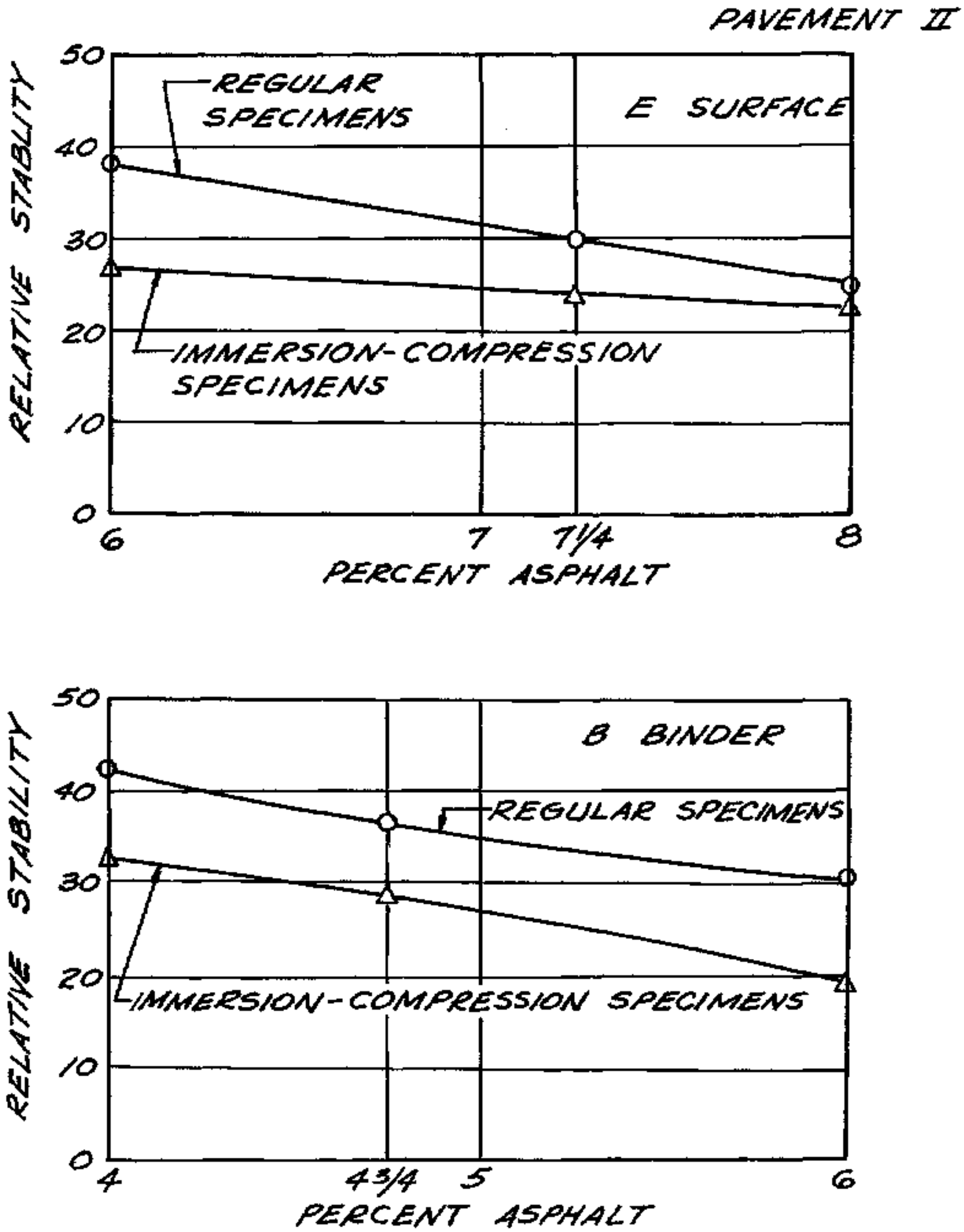


Figure C2. Variation in Relative Stability with Asphalt Content for Pavement II.

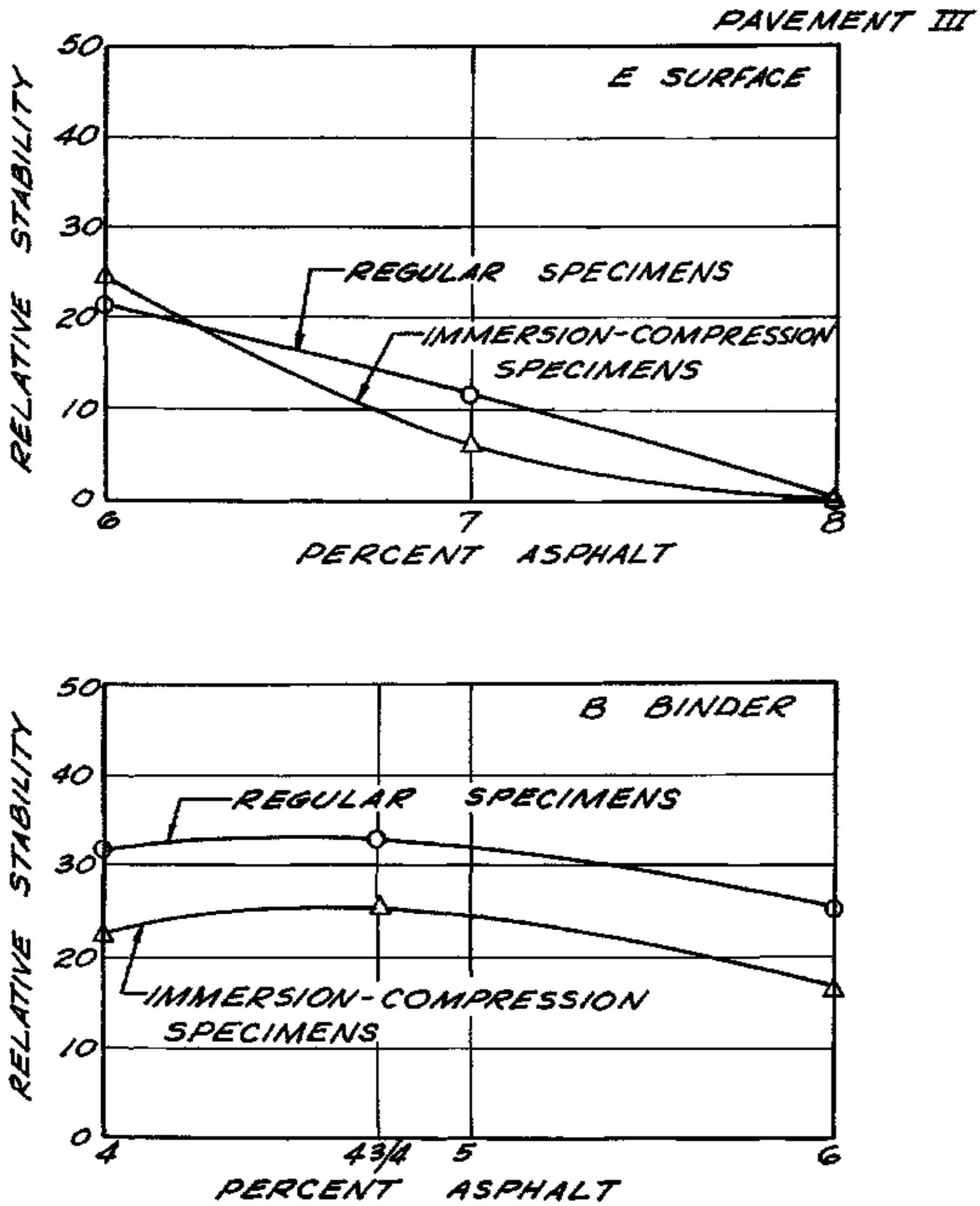


Figure C3. Variation in Relative Stability with Asphalt Content for Pavement III.

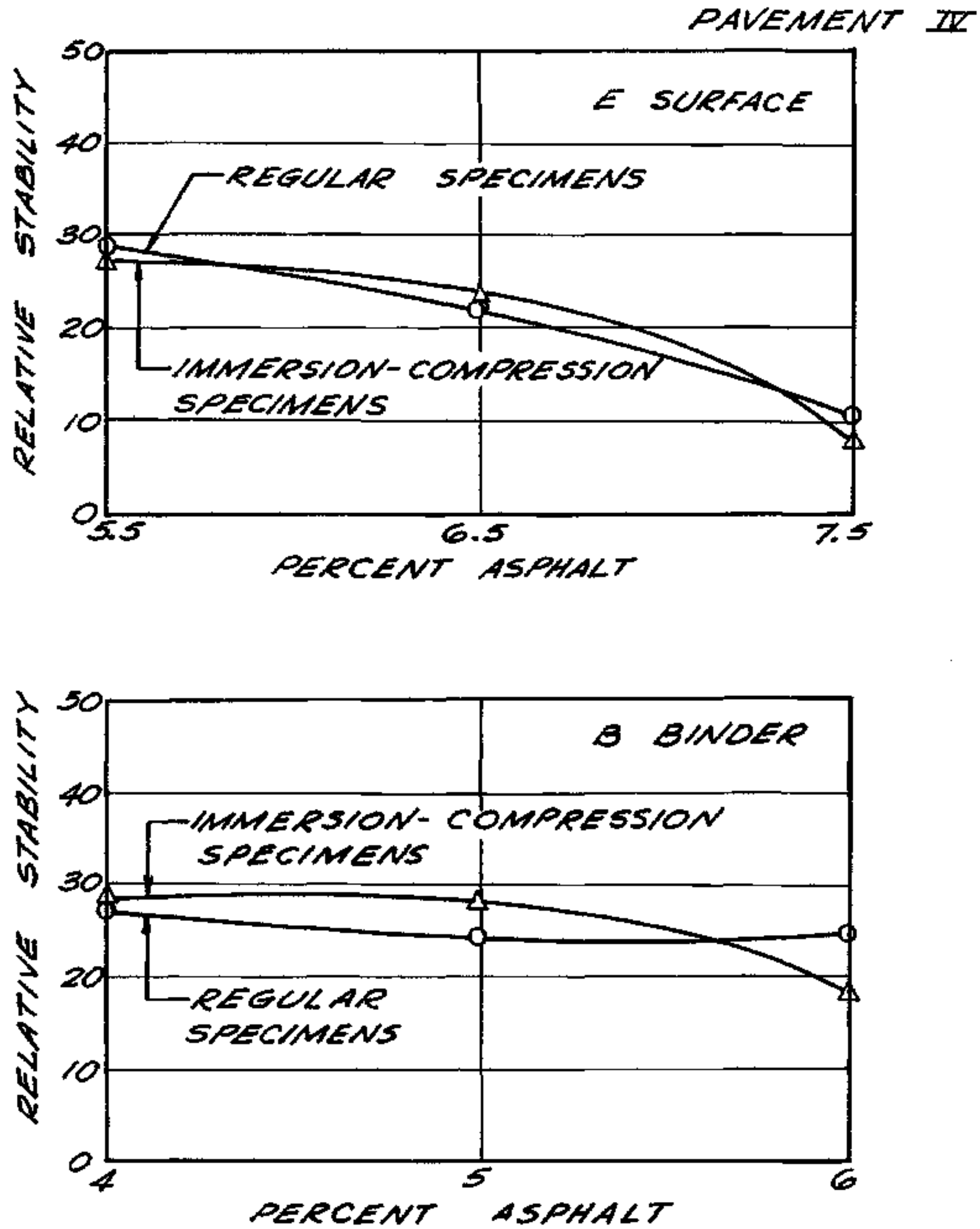


Figure C4. Variation in Relative Stability with Asphalt Content for Pavement IV.

APPENDIX D

RESULTS OF TRIAXIAL SHEAR TESTS

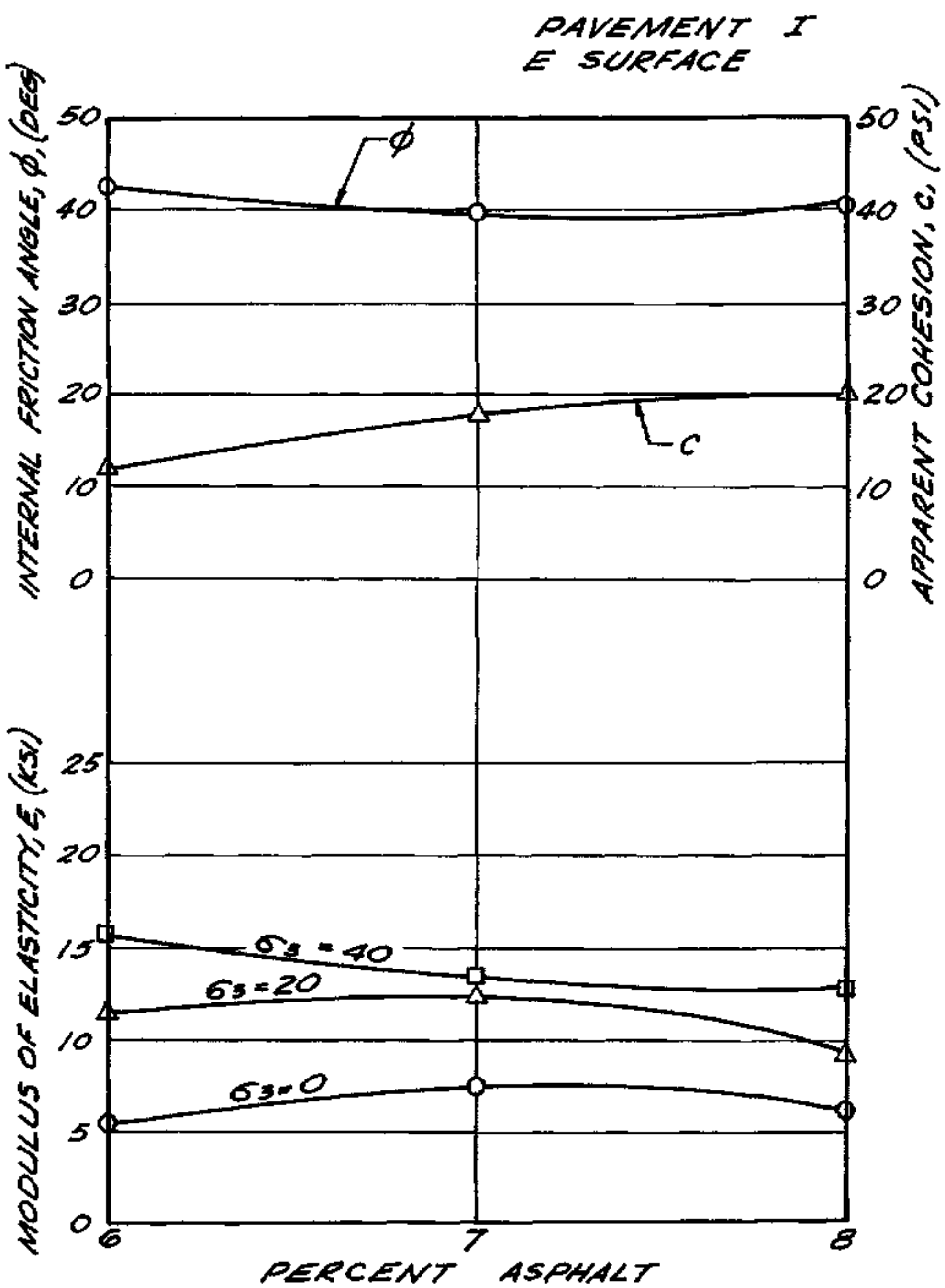


Figure D1. Variation in Internal Friction Angle, Apparent Cohesion, and Modulus of Elasticity with Asphalt Content for E Surface, Pavement I.

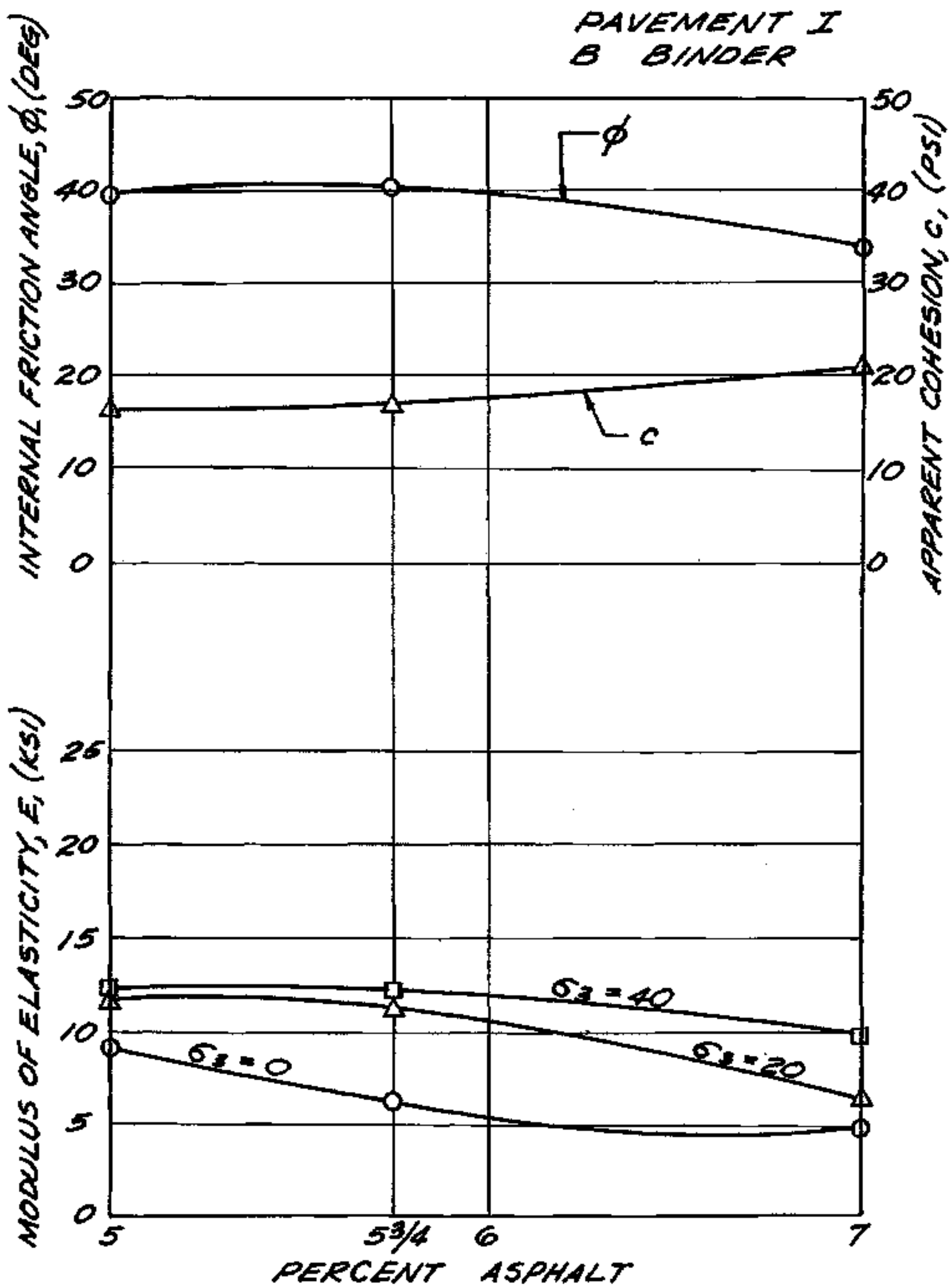


Figure D2. Variation in Internal Friction Angle, Apparent Cohesion, and Modulus of Elasticity with Asphalt Content for B Binder, Pavement I.

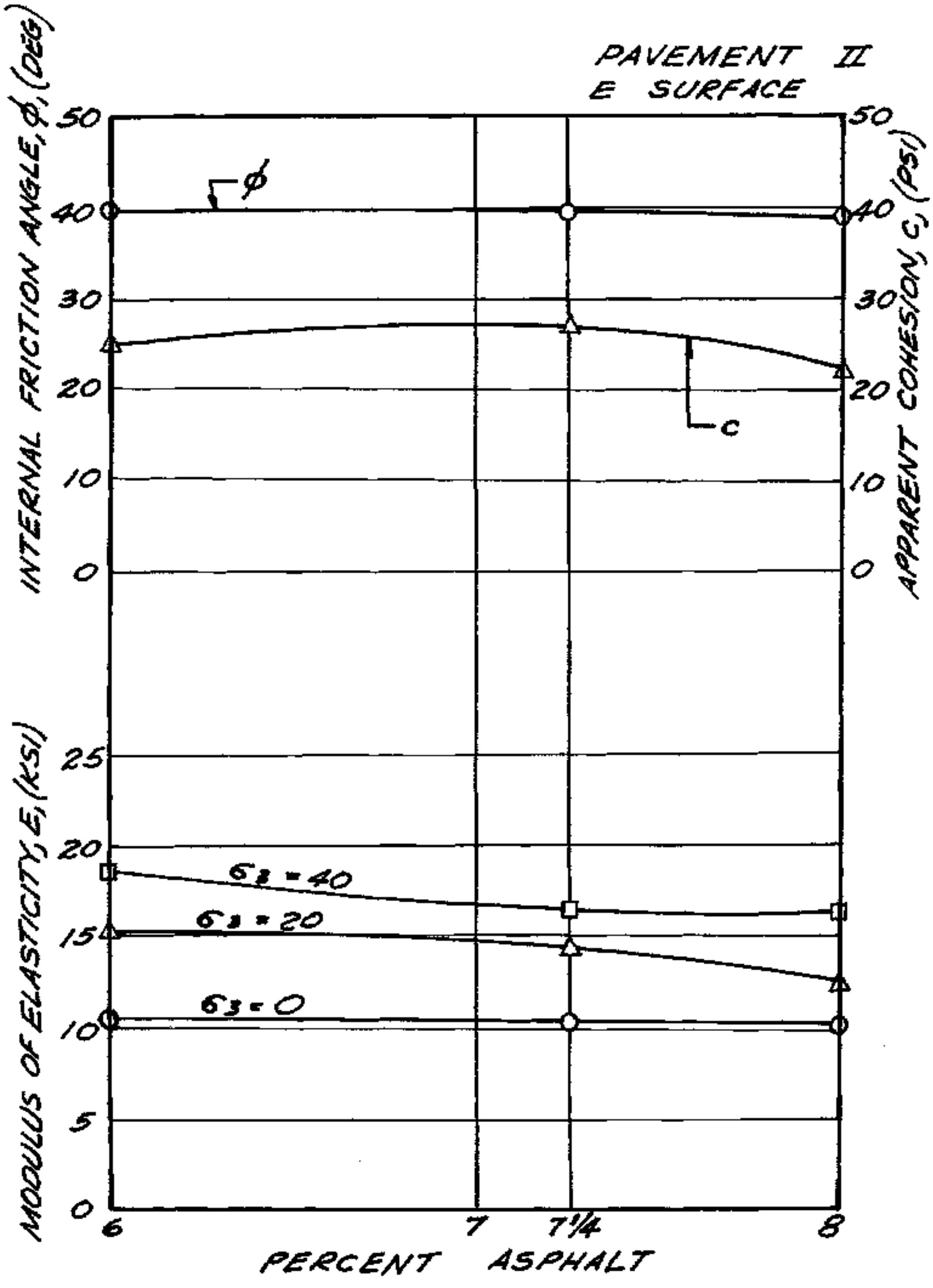


Figure D3. Variation in Internal Friction Angle, Apparent Cohesion, and Modulus of Elasticity with Asphalt Content for E Surface, Pavement II.

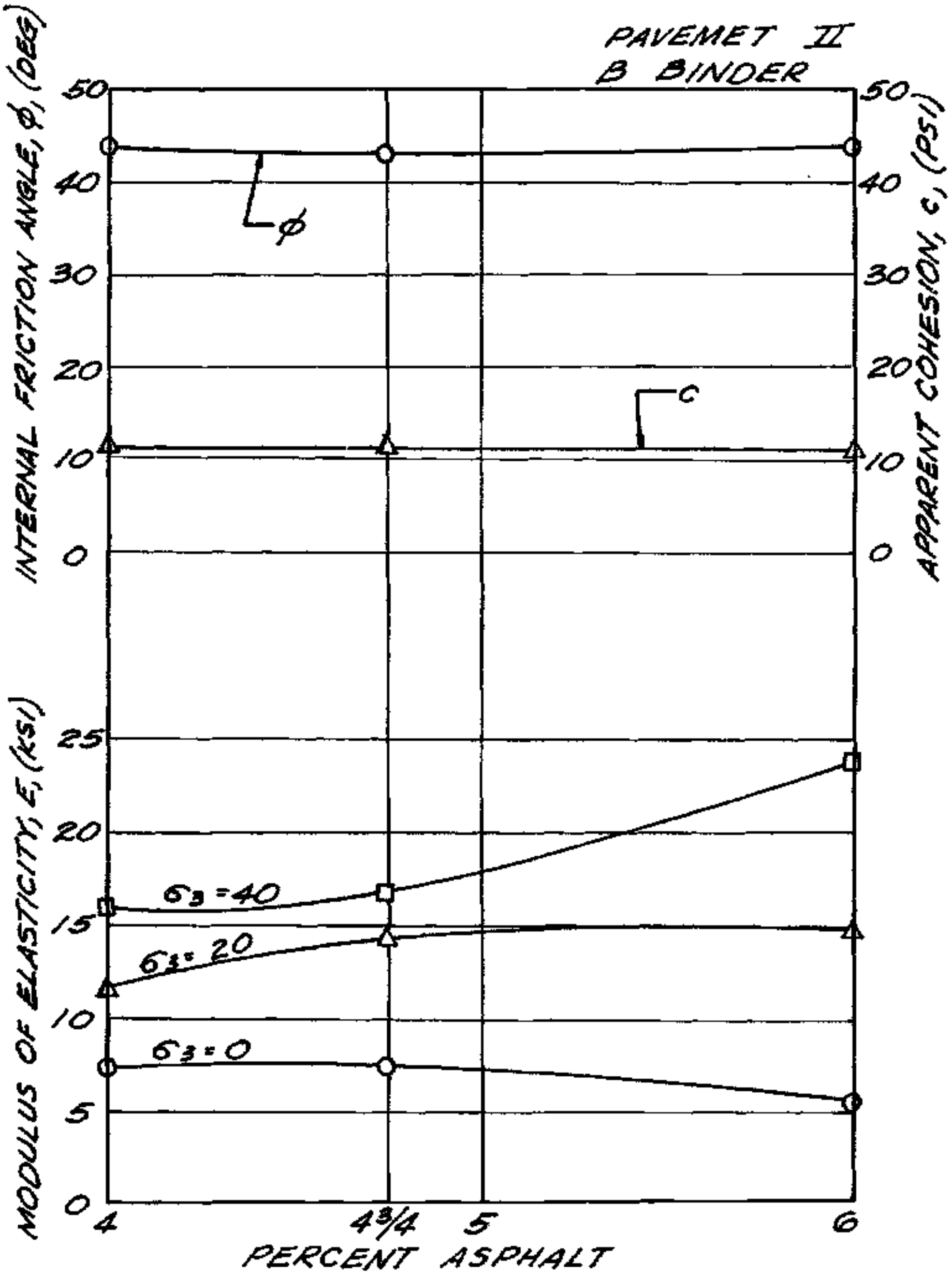


Figure D4. Variation in Internal Friction Angle, Apparent Cohesion, and Modulus of Elasticity with Asphalt Content for B Binder, Pavement II.

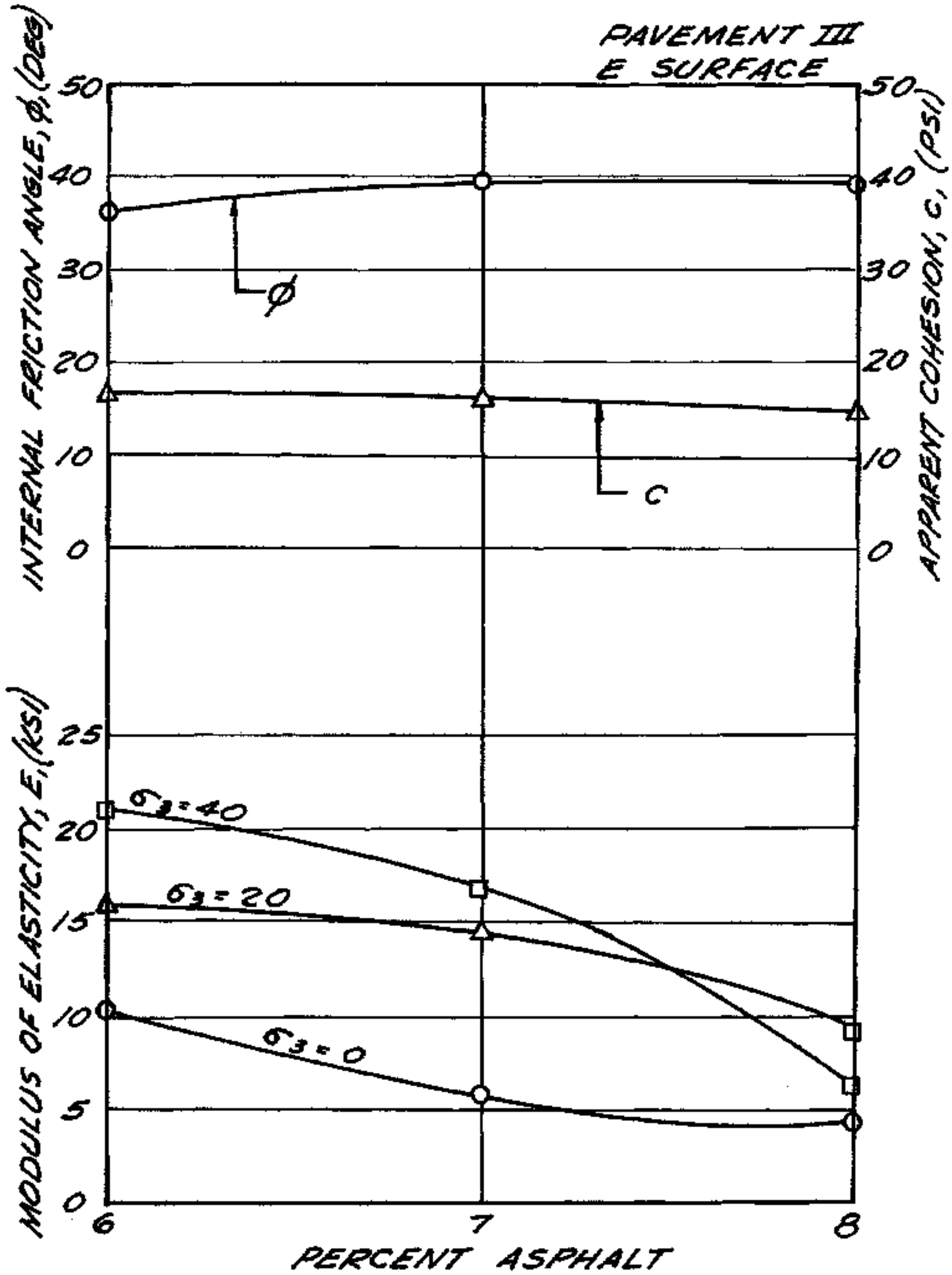


Figure D5. Variation in Internal Friction Angle, Apparent Cohesion, and Modulus of Elasticity with Asphalt Content for E Surface, Pavement III.

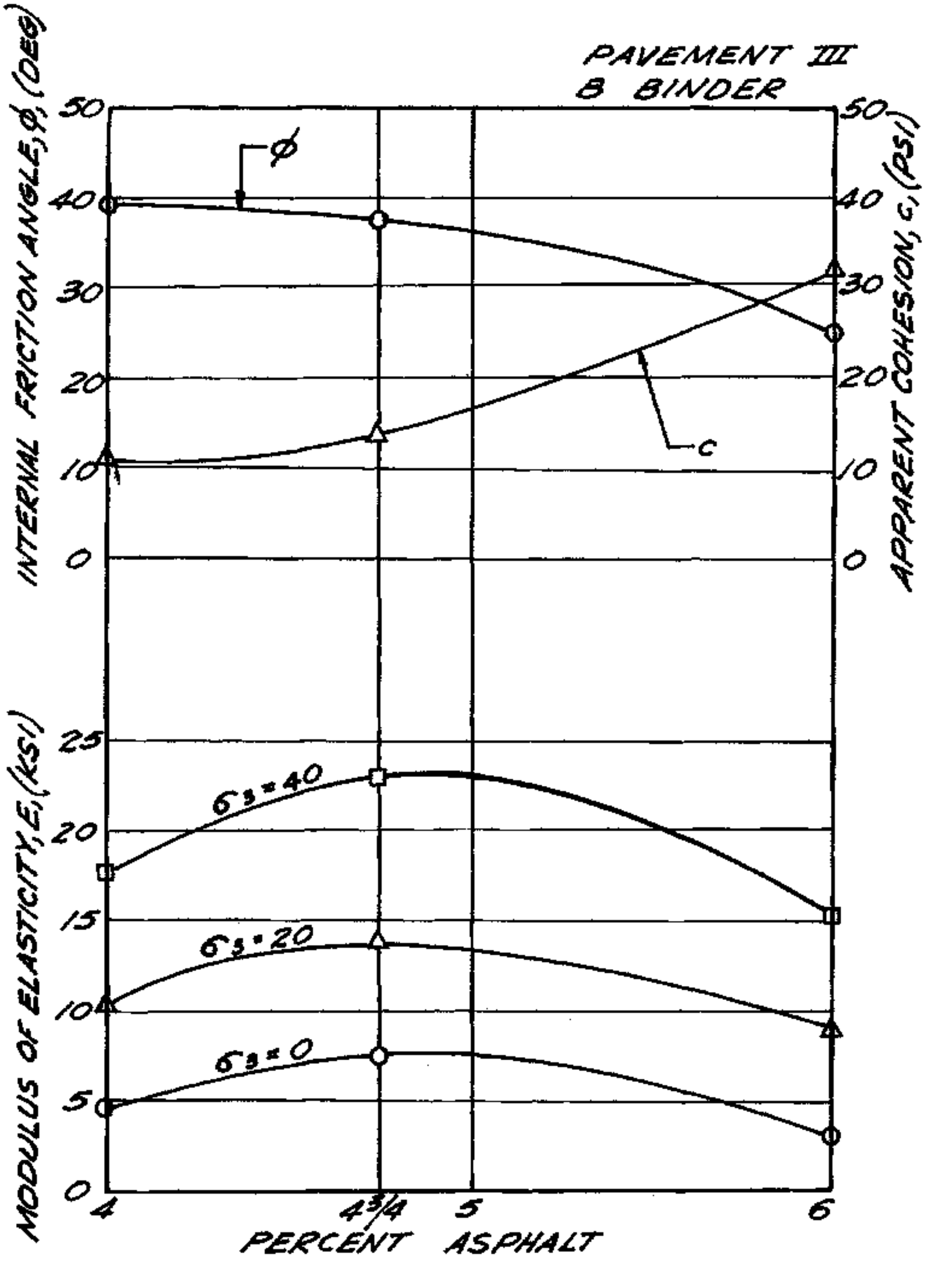


Figure D6. Variation in Internal Friction Angle, Apparent Cohesion, and Modulus of Elasticity with Asphalt Content for B Binder, Pavement III.

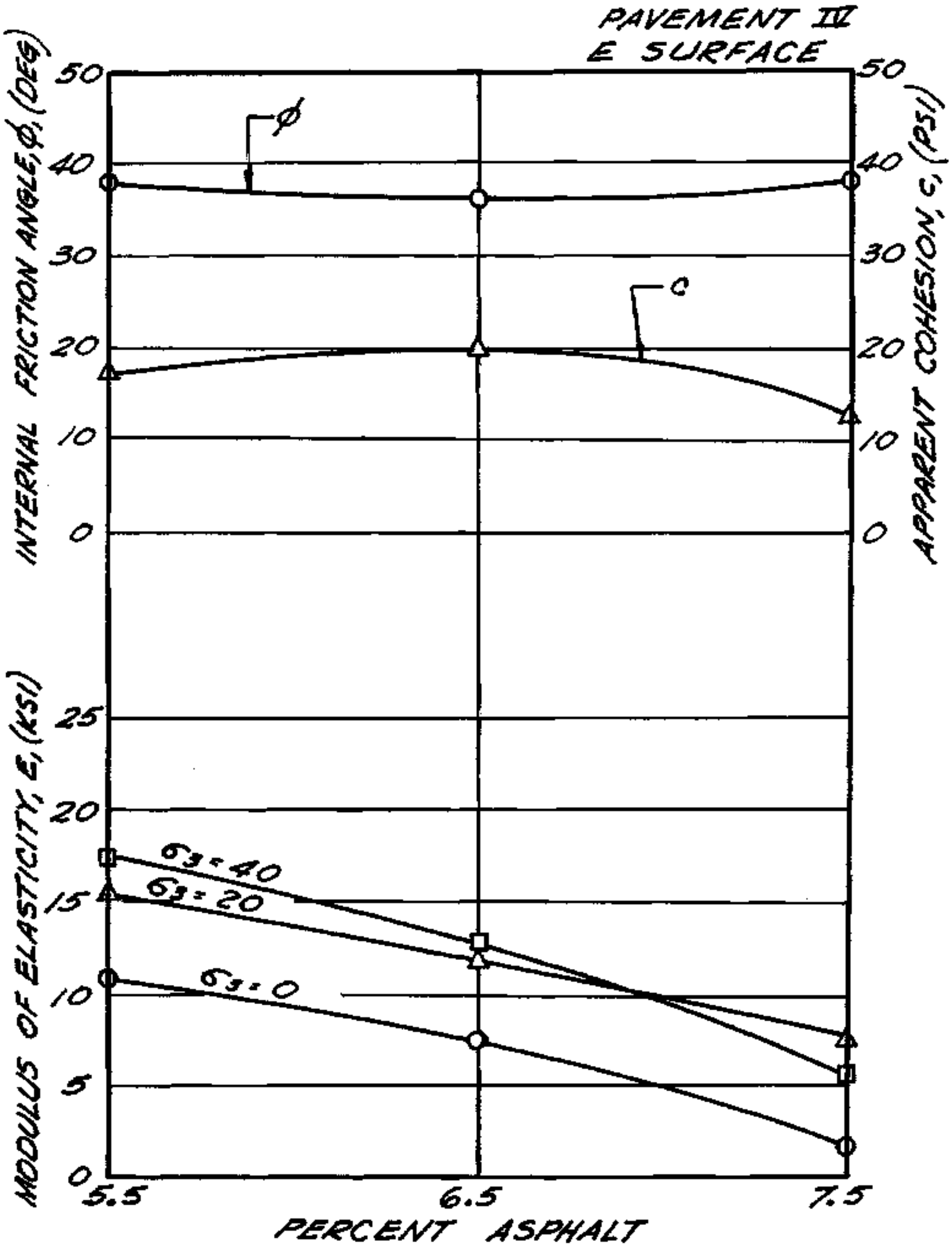


Figure D7. Variation in Internal Friction Angle, Apparent Cohesion, and Modulus of Elasticity with Asphalt Content for E Surface, Pavement IV.

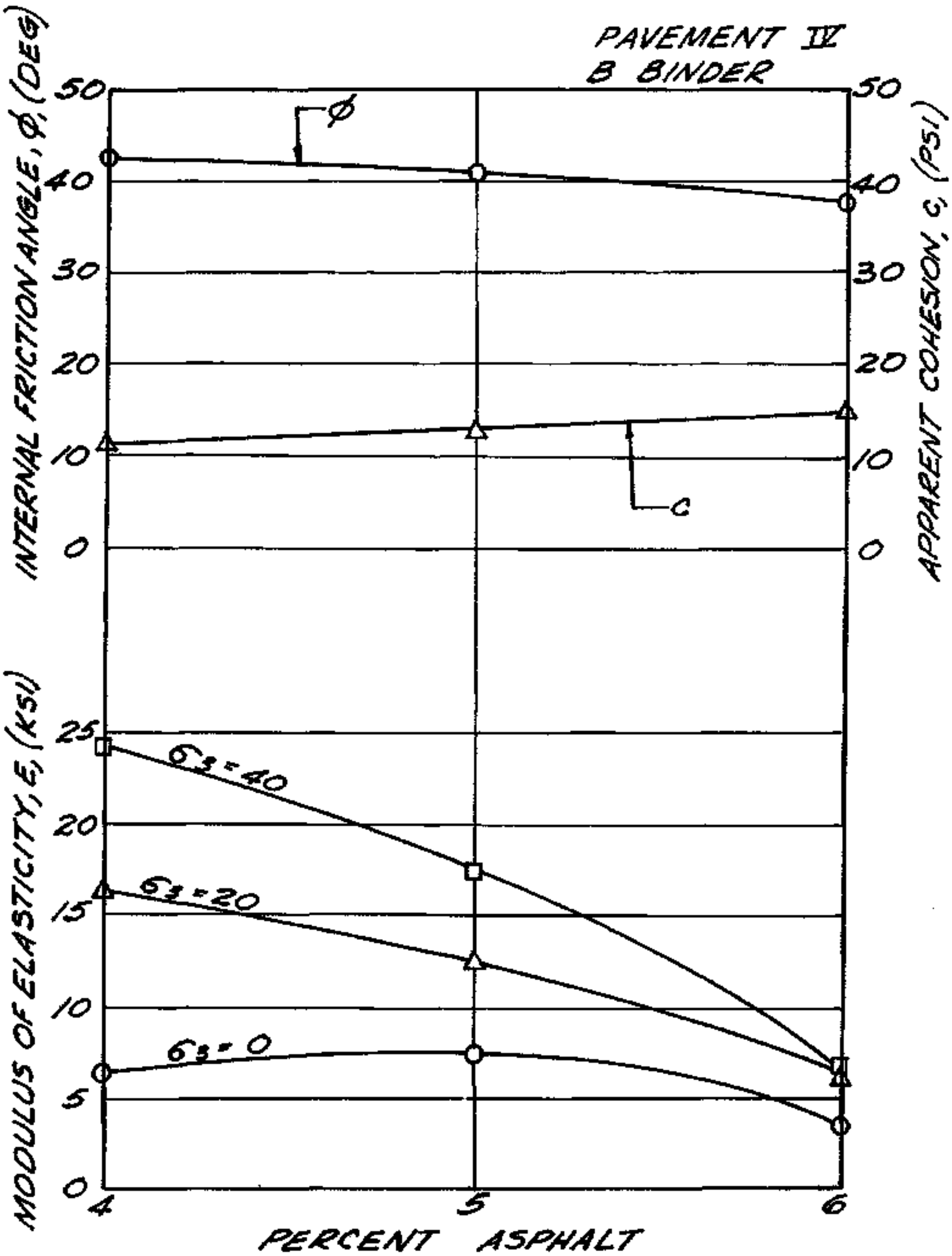


Figure D8. Variation in Internal Friction Angle, Apparent Cohesion, and Modulus of Elasticity with Asphalt Content for B Binder, Pavement IV,

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